California High-Speed Rail Authority

RFP No.: HSR 13-57

Request for Proposals for Design-Build Services for Construction Package 2-3

Reference Material, Part C.11
PE4P Non-standard and Complex Structures Report

RFP No.: 13-57 – Addendum No. 2 - 06/30/2014

CALIFORNIA HIGH-SPEED TRAIN

Engineering Report

Preliminary Engineering for Procurement Record Set Submission

Fresno to Bakersfield

Sierra Subdivision Construction Package 2-3 Non-standard and Complex Structures Report

June 2014

RFP No.: 13-57 – Addendum No. 2 - 06/30/2014

CALIFORNIA
High-Speed Rail Authority

Record Set
Fresno to Bakersfield
Sierra Subdivision
PE4P Construction Package 2-3
Nonstandard and Complex
Structures Report

RFP No.: 13-57 – Addendum No. 2 - 06/30/2014

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URS/HMM/Arup Joint Venture

June 2014

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List of Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
Authority	California High-Speed Rail Authority
Caltrans	California Department of Transportation
CHSTP	California High-Speed Train Project
CIDH	cast-in-drilled-hole
CP	Construction Package
CSDC	Caltrans Seismic Design Criteria
CSiBridge	CSiBridge V1520 (Computers and Structures, Inc.)
CVRR	Central Valley Railroad
EIS	environmental impact statement
FB	Fresno to Bakersfield
GDR	Geotechnical Data Report
GI	ground investigation
HSR	high-speed rail
KTR	Kings/Tulare Regional (Station)
LLRM	modified Cooper E-50 loading
LLRR	maintenance and construction train (Cooper E-50)
MCE	maximum considered earthquake
MSE	mechanically stabilized earth
NCL	no-collapse performance level
NFPA	national fire protection association
OBE	operating basis earthquake
OPL	operability performance level
PE4P	preliminary engineering for procurement
PC	prestressed concrete
RC	reinforced concrete
SAP	Structural Analysis Program 2000 V14 (Computers and Structures Inc.)
SR	State Route
SJVR	San Joaquin Valley Railroad
TM	Technical Memorandum
USACE	United States Army Corps of Engineers

Section 1.0

Introduction

1.0 Introduction

1.1 Project Overview

In 1996, the State of California established the California High-Speed Rail Authority (Authority). The Authority is responsible for studying alternatives to construct a rail system that will provide intercity high-speed rail (HSR) service on over 800 miles of track throughout California. This rail system will connect the major population centers of Sacramento, the San Francisco Bay Area, the Central Valley, Los Angeles, the Inland Empire, Orange County, and San Diego. The Authority is coordinating the project with the Federal Railroad Administration. The California High-Speed Train Project (CHSTP) is envisioned as a state-of-the-art, electrically powered, high-speed, steel-wheel-on-steel-rail technology that will include state-of-the-art safety, signaling, and automated train-control systems.

The statewide CHSTP has been divided into a number of sections for the planning, environmental review, coordination, and implementation of the project. This *Nonstandard and Complex Structures Report* is focused on the section of the CHSTP between Fresno and Bakersfield, specifically the Construction Package (CP) 2-3 subsection of the alignment extending from E American Avenue south of the Fresno metropolitan area to 1 mile north of the Tulare/Kern County line.

1.2 Project Description

1.2.1 Fresno to Bakersfield High-Speed Rail Section

The proposed Fresno to Bakersfield (FB) Section of the HSR is approximately 114 miles long and traverses a variety of land uses, including farmland, large cities, and small cities. The FB Section includes viaducts and segments where the HSR will be at-grade or on embankment. The route of the FB Section passes by or through the rural communities of Bowles, Laton, Armona, and Allensworth and the cities of Fresno, Hanford, Selma, Corcoran, Wasco, Shafter, McFarland, and Bakersfield.

The FB Section extends from north of Stanislaus Street in Fresno to the northern most limit of the Bakersfield to Palmdale Section of the HSR at Oswell Street in Bakersfield.

CP2-3 begins at STA 587+30.67 adjacent to E American Avenue to the south of Fresno and finishes at STA 4435+50, 1 mile North of the Tulare/Kern County line and approximately at the midpoint of Allensworth Bypass (Stationing Ref to 15% Record Set).

1.2.2 Alignments

The FB HSR Section, shown in Figure 1.2-1, is a critical link connecting the northern HSR sections of Merced to Fresno and the Bay Area to the southern HSR sections of Bakersfield to Palmdale and Palmdale to Los Angeles. The FB Section includes HSR stations in the cities of Fresno and Bakersfield, with provision for constructing a third station near Hanford subject to achieving ridership targets. The Fresno and Bakersfield stations are this section's project termini.

The FB Section of the HSR is generally divided into the following subsections with alignment prefixes. Table 1.2-1, Figure 1.2-1 and Figure 1.2-1 and illustrate the subsections and their corresponding alignments.

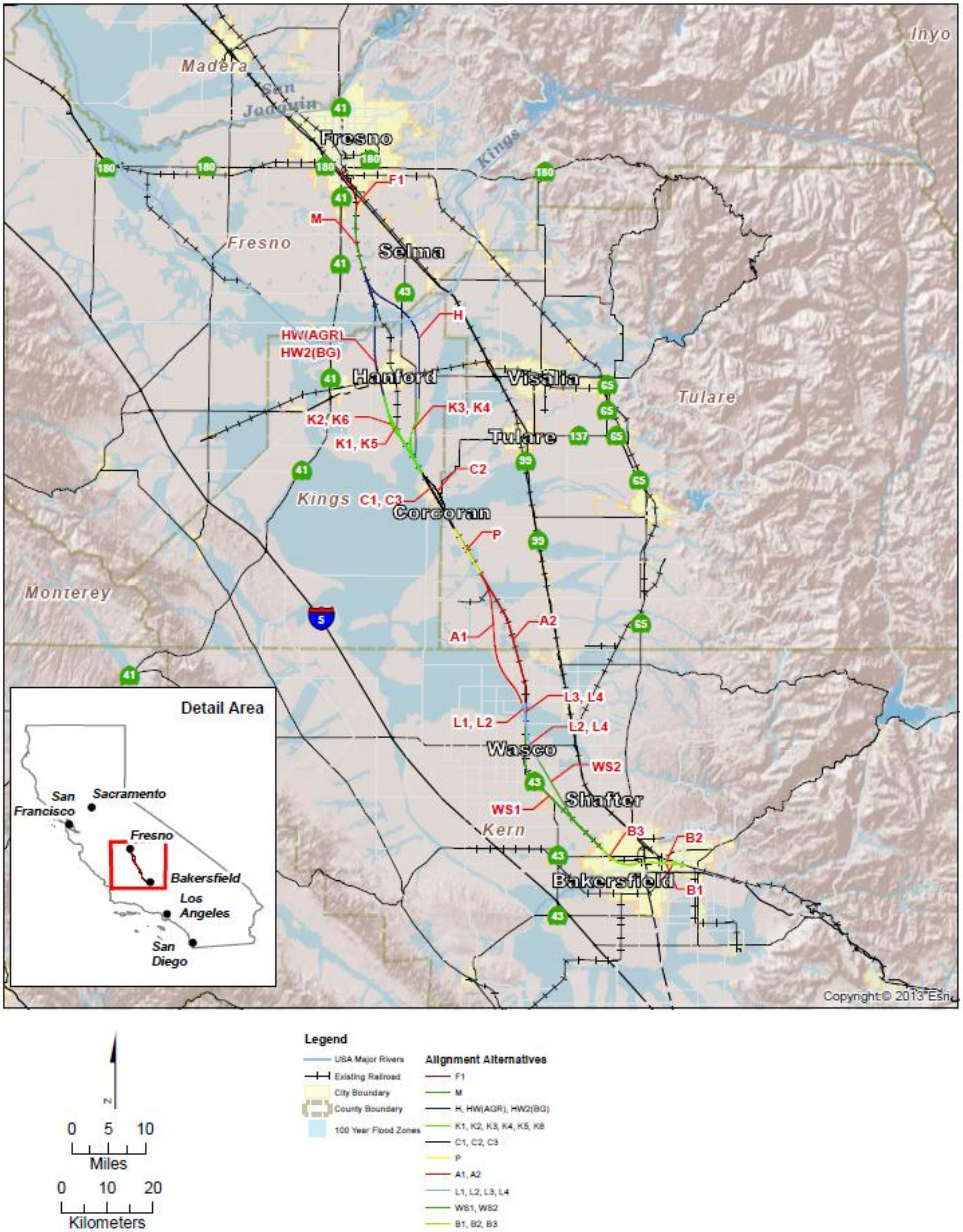

Figure 1.2-1
Overview of Alignments

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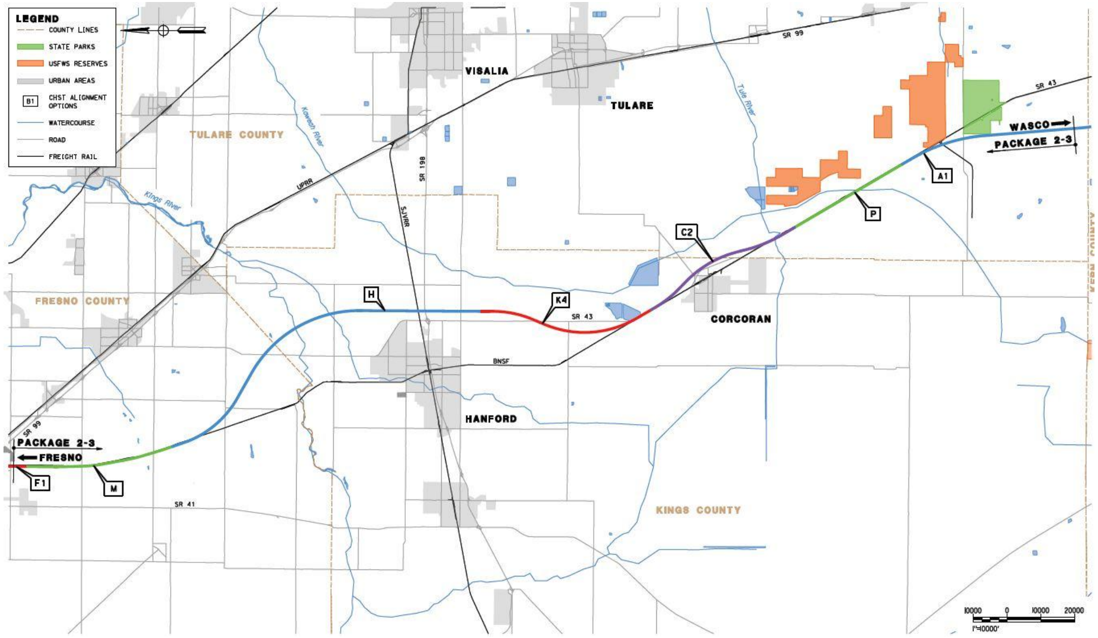

Figure 1.2-2
Overview of CP2-3 Section

Table 1.2-1
FB Alignment Subsections in CP2-3

Alignment Prefix	Alignment Subsection Name	Location		County	EIR/EIS* Name
		Begin	End		
F1	Fresno	Stanislaus St	E Jefferson Ave	Fresno	BNSF
M	Monmouth	E Lincoln Ave	E Kamm Ave	Fresno	BNSF
H	Hanford	E Kamm Ave	Iona Ave	Fresno and Kings	BNSF (Hanford East)
K4	Kaweah	Iona Ave	Nevada Ave	Kings	BNSF (Hanford East) (connects to C2 [Corcoran Bypass])
C2	Corcoran Bypass	Nevada Ave	Ave 128	Kings	Corcoran Bypass
P	Pixley	Ave 128	Ave 84	Tulare	BNSF
A1	Allensworth Bypass	Ave 84	Elmo Hwy	Tulare	Allensworth Bypass

*Environmental Impact Report/Statement

1.2.3 Structures

Of the 114-mile FB Section, as much as 30% of the HSR mainline will be carried on structure. Alignments are typically elevated to clear obstacles such as existing railroads, roadways, and waterways, but elevated structures may also be proposed in floodways or as an effort to reduce impacts on nearby properties.

The majority of elevated structures will be in the form of aerial viaducts, composed of a standard design of prestressed concrete box girders. In locations where it is not practical to use the standard box girder type, other structural types have been proposed, such as trusses, balanced cantilevers, and elevated slabs. The reasoning for using each type has been discussed in the *15% Record Set Advance Planning Study Report* and the *15% Record Set Constructability Assessment Memo.*

In circumstances where the proposed mainline will disrupt existing infrastructure routes, such as existing roadway networks, new structures are proposed to allow these networks to maintain connectivity over the HSR right-of-way. Preliminary roadway realignments and roadway structure designs have been developed as part of the 15% design phase.

In addition to the defined roadway and HSR mainline structures, several ancillary structures have been addressed as part of the preliminary design. Most of these structures have been identified in order to service existing railroad lines that will be affected by the proposed HSR alignment, most notably the BNSF Railroad and the San Joaquin Valley Railroad.

1.3 Structure Classification

This report covers only the HSR structures within CP2-3 considered to be nonstandard or complex. TM2.3.2 Structure Design Loads R2 (April 20, 2011) defines the following hierarchy of structure types:

- Primary structures: structures that directly support the HSR tracks.
- Secondary structures: all other structures.

Primary structures are subdivided by importance into the following:

- Important structures: structures designated by the Authority to be important.
- Ordinary structures: all other structures.

Primary structures are also classified by technical complexity in TM 2.10.4 Seismic Design Criteria R1 (May 26, 2011) as follows:

- Complex structures: Structures that have complex response during seismic events through
  - Irregular geometry.
  - Unusual framing.
  - Long spans.
  - Unusual geologic conditions.
  - Close proximity to hazardous faults.
  - Regions of severe ground motion.
- Standard structures: Structures that are not Complex structures and comply with the pending CHSTP Guidelines for Standard Aerial Structures.
- Nonstandard structures: Structures that do not meet the requirements for either Standard or Complex structures.

Table 1.3-1 lists the structures in CP2-3 of the FB Section and indicates their classification under the above system. All the listed structures directly support the HSR and are classified as Primary.

Table 1.3-1
Mainline Structure Key Data and Classification

No.	Purpose (e.g., span over river, local roads)	Alignment	Location (beg. station)	Structural Type (e.g., balanced cantilever)	Length (ft)	Max. Column Height (ft)	No. of Bents	No. of Spans	Clearances to Local Facilities	Structure Classification
1	Retaining Wall	H	1086+00	MSE Wall (Retained Embankment)	1,970		N/A	N/A	N/A	Nonstandard
2	Viaduct crossing E Conejo Ave	H	1105+70	PC Girder	1,460		13	13	N/A	Standard
3	**Conejo Crossover Structure**	H	1120+30	Crossover Beam/Slab Structure	1,429		N/A	N/A	27ft-4in to BNSF	Nonstandard
4	Viaduct crossing S Peach Ave	H	1134+60	PC Girder	2,160		17	17	17ft-7in to S Peach Ave	Standard
5	Retaining Wall	H	1156+20	MSE Wall (Retained Embankment)	1,730		N/A	N/A	N/A	Nonstandard
6	Retaining Wall	H	1439+19	MSE Wall (Retained Embankment)	2,439		N/A	N/A	N/A	Nonstandard
7	Kings River Viaduct	H	1463+58	PC Girder	121.5	16	1	1		Standard
8	Kings River Viaduct‡	H	1464+80	Truss Span	217	N/A*	1	1	18ft-0in to SR 43	Complex
9	Kings River Viaduct	H	1466+97	PC Girder	1,863	17	15	15		Standard
10	Cole Slough Bridge‡	H	1485+60	Steel Truss	357		1	1	18ft-4in to levee	Complex
11	Kings River Viaduct	H	1489+17	PC Girder	2,903	17	25	25		Standard
12	**Dutch John Cut**	H	1518+20	Steel Truss	714		1	2	N/A	Complex
13	Kings River Viaduct	H	1525+33	PC Girder	5,584	17	47	47	16ft-8in Ninth Ave 17ft-0in Cairo Ave	Standard

No.	Purpose (e.g., span over river, local roads)	Alignment	Location (beg. station)	Structural Type (e.g., balanced cantilever)	Length (ft)	Max. Column Height (ft)	No. of Bents	No. of Spans	Clearances to Local Facilities	Structure Classification
14	Kings River Bridge‡	H	1581+17	Steel Truss	644		1	2	N/A	Complex
15	Kings River Viaduct	H	1587+60	PC Girder	603	17	5	5		Standard
16	Levee Road Bridge‡	H	1593+64	Steel Truss	283.5		1	1	19ft-5in Levee Road	Complex
17	Retaining Wall	H	1596+51	MSE Wall (Retained Embankment)	2,599		N/A	N/A	N/A	Nonstandard
18	Retaining Wall	H	1885+40	MSE Wall (Retained Embankment)	1,817		N/A	N/A	N/A	Nonstandard
19	**Hanford Viaduct (including Kings/Tulare Regional Station)**	H	1903+57	PC Girder, precast standard spans, precast nonstandard spans	10,480	40	86	86	Grangeville Boulevard 23ft-2in Cross Valley Railroad 32ft-6in Lacey Boulevard 29ft-4in SR 198 25ft-9in	Nonstandard & Standard
20	Retaining Wall	H	2008+37	MSE Wall (Retained Embankment)	1,511		N/A	N/A	N/A	Nonstandard
21	**Kaweah SR 43 Crossing**	K4	2240+32	Steel Truss	574		1	2	16ft-6in to SR 43	Complex
22	Retaining Wall	K4	2436+00	MSE Wall (Retained Embankment)	1,081		N/A	N/A	N/A	Nonstandard
23	Cross Creek Viaduct	K4	2446+81	PC Girder	3,241		28	28		Standard
24	Cross Creek Bridge‡	K4	2479+22	Steel Truss	322		0	1		Complex
25	Cross Creek Viaduct	K4	2482+44	PC Girder	4,741.5		41	41		Standard
26	Cross Creek Viaduct‡	K4	2530+00	Crossover Beam/Slab Structure	525	18ft-6in	N/A	N/A	18ft-5in to SR 43	Nonstandard

No.	Purpose (e.g., span over river, local roads)	Alignment	Location (beg. station)	Structural Type (e.g., balanced cantilever)	Length (ft)	Max. Column Height (ft)	No. of Bents	No. of Spans	Clearances to Local Facilities	Structure Classification
27	Cross Creek Viaduct	K4	2535+11	PC Girder	360		3	3		Standard
28	Retaining Wall	K4	2538+71	MSE Wall (Retained Embankment)	4,492	N/A	N/A	N/A	N/A	Nonstandard
29	Whitley Ave/SR 137	C2	2812+76	Steel Half Through Girder	90	N/A	0	1	22ft-10in Whitley Ave	Non Standard
30	Retaining Wall	C2	2966+50	MSE Wall (Retained Embankment)	2,286	N/A	N/A	N/A		Nonstandard
31	SR 43/BNSF Viaduct	C2	2989+36	PC Girder	1,564		13	13	21ft-9in Popular Ave	Standard
32	**Corcoran Crossover Structure (part of SR 43/BNSF Viaduct)**	C2	3005+00	Crossover Beam/Slab Structure	2,426		N/A	N/A	24ft-8in SR 43 27ft-3in BNSF	Nonstandard
33	SR 43/BNSF Viaduct	C2	3029+21	PC Girder	1,681		14	14		Standard
34	♦ Retaining Wall	C2	3046+02	MSE Wall (Retained Embankment)	1,868		N/A	N/A		Nonstandard
35	♦ Retaining Wall	A1	3982+20	MSE Wall (Retained Embankment)	2,305		N/A	N/A		Nonstandard
36	Deer Creek Viaduct	A1	4005+25	PC Girder	6,240		54	54	24ft-6in Stoil Spur	Standard
37	♦ Retaining Wall	A1	4067+65	MSE Wall (Retained Embankment)	1,830		N/A	N/A		Nonstandard

*N/A indicates where information required is not applicable to the structure, e.g., bents and spans for retaining walls.
Note: Not all nonstandard and complex structures are discussed in this report. Bolded titles denote those structures that are discussed further.
‡ indicates structures that are similar in concept to the structures discussed in this report.

1.4 Overall Design Assumptions for Preliminary Engineering

In carrying out the analysis of complex and nonstandard structures, the Regional Consultant has concentrated on the key aspects of the design stated in the analysis scope. These aspects are determined in many cases by satisfying the requirements of the relevant design criteria.

For the bridge structures, the requirements include the following:

- Structural adequacy.
- Seismic performance as specified in the seismic design criteria.
- Interaction between track and structure to verify the compatibility of the structure and track systems.
- Constructability and assumed construction method.
- Design economy.

1.4.1 Structure Descriptions

The Regional Consultant has identified the following complex and nonstandard structures as representative examples of the structure types within CP2-3 of the CHSTP:

- Conejo Crossover Structure.
- Dutch John Cut Bridge.
- Kings/Tulare Regional Station.
- Kaweah State Route (SR) 43 Crossing.
- Corcoran Crossover Structure.

These structures have been the subject of detailed analysis to determine their capability for further development into compliant designs.

The Conejo Crossover Structure is the section of the Conejo Viaduct that crosses the BNSF line. On the east end of the crossover structure is the E Conejo Avenue standard viaduct. To the west is the S Peach Avenue standard viaduct (Rows 2, 3, and 4 in Table 1.3-1). The crossover structure is a large structure that is conceived as a slab supported on multiple columns to either side of the BNSF railroad corridor. The slab section is assumed to be constructed by placing precast beams across the railway supported on deep in situ concrete column cap beams that run parallel to the railway. The 6-foot-diameter columns are positioned at 30-foot centers along the length of the structure and are founded on a single 9-foot diameter pile of approximately 170 feet in depth.

Dutch John Cut Bridge (Row 12 in Table 1.3-1) is a two-span truss structure that crosses one channel of Kings River. As the channel capacity is controlled by levees to limit flooding, the structure has been designed to provide a minimum maintenance clearance to the top of the levee of 18 feet which has been agreed in discussion with the Kings River Conservation District (see *Hydrology, Hydraulics and Drainage Report*). Additionally the structure over-spans the levee with two 350-foot spans to avoid issues with permitting.

Kings/Tulare Regional Station (Hanford) (Row 19 of Table 1.3-1) has been modeled as a series of in situ concrete post-tensioned girders to accommodate areas of the alignment where turnout switches are required. The structure provides additional space beside the turnouts which is potentially useful as laydown space for equipment associated with track and switch maintenance. The location of the turnouts dictates areas of the structure where joints are not permitted and which therefore determines the span configuration to be used.

The Kaweah SR 43 Crossing (Row 21 in Table 1.3-1) is a steel truss bridge of two spans. This form was chosen to minimize the depth of the underpass cutting and also to allow the route to cross SR 43 at high skew, avoiding extensive realignment of the highway.

The Corcoran Crossover Structure (Row 32 in table 1.3-1) is similar to the Conejo Crossover structure but has two spans and is located in a higher seismic zone.

These structures have been modeled using either SAP2000 V14 (SAP)/CSiBridge V1520 (CSiBridge) or MIDAS to confirm the structural adequacy of the concept. Other structures such as roadway crossings have not been modeled as they present no specific technical difficulties.

1.4.2 Seismic Performance

The seismic design criteria specified in TM 2.10.4 provide requirements for assessment of the seismic performance of structures. The seismic design criteria define the two design-level earthquakes as follows:

- Maximum considered earthquake (MCE) – ground motions corresponding to greater of
  - (1) a probabilistic spectrum based upon a 10% probability of exceedence in 100 years (i.e., a return period of 950 years with 5% damping) and
  - (2) a deterministic spectrum based upon the largest median response resulting from the maximum rupture (corresponding to Mw) of any fault in the vicinity of the structure.
- Operating basis earthquake (OBE) – Ground motions corresponding to a probabilistic spectrum based upon an 86% probability of exceedence in 100 years (i.e., a return period of 50 years with 5% damping).

In terms of acceptability of the design, the requirements relating to seismic performance are the operability performance level (OPL) under the action of the OBE and no-collapse performance level (NCL) under the action of the MCE.

These performance levels imply the following:

- OPL at OBE.
  - Minimal impacts to HSR operations.
  - No spalling of concrete.
  - Minimal permanent deformations.
- NCL at MCE.
  - No collapse.
  - Significant yielding of reinforcing steel.
  - Extensive cracking and spalling of concrete but minimal loss of vertical load carrying capacity in columns.
  - Large permanent deflections.

Response spectra for design of the route from Merced to Bakersfield have been the subject of a separate study (Kleinfelder – Ground Motions for Preliminary Design of California High Speed Rail Project, Seismic Ground Motion Zone Map). The design spectra relevant to CP2-3 lie partly within Zone 4 and partly within Zone 5. The acceleration response spectrum curves used for preliminary design are reproduced in Figure 1.4-1 and Figure 1.4-2. It is expected that finalized acceleration response spectrum curves and design criteria will be provided to contractors for the subsequent design development stages.

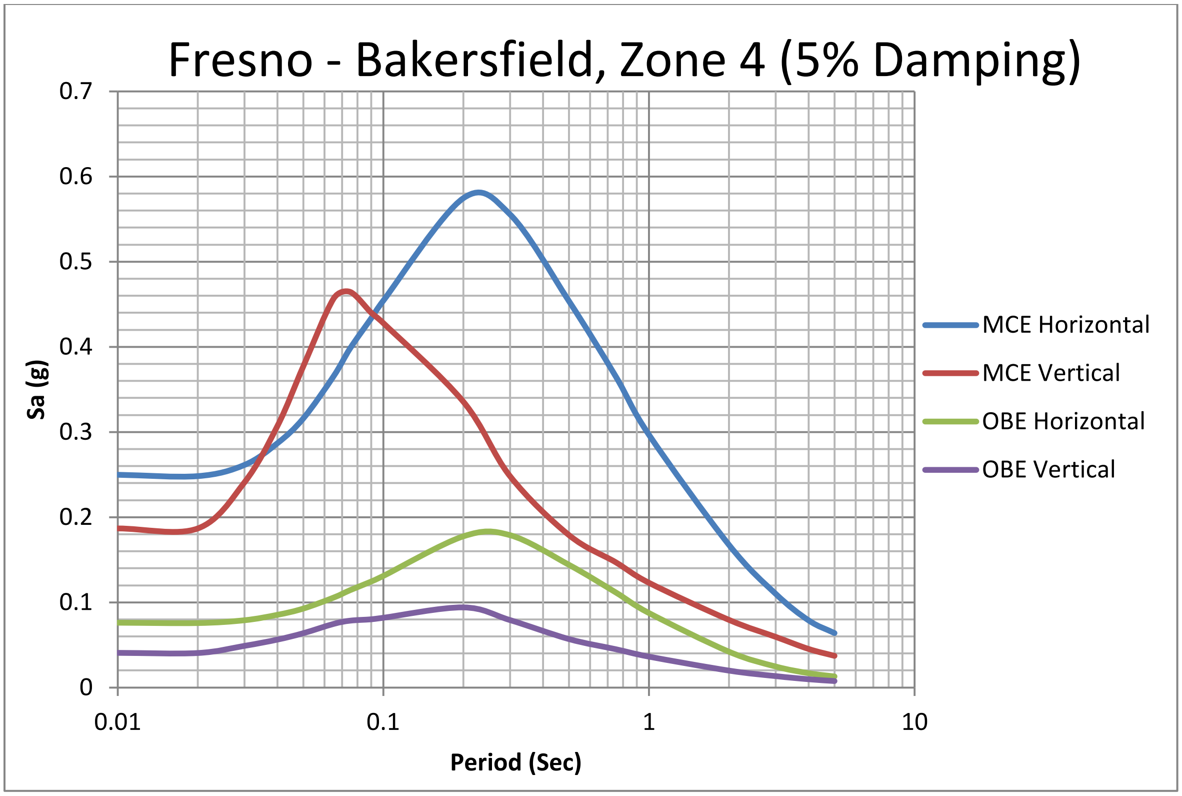

Figure 1.4-1
Design Response Spectra (Zone 4)

For Zone 4, the peak horizontal ground acceleration has been taken as the acceleration that corresponds to a period of 0.01 seconds — that is 0.0761g at OBE (green curve) and 0.2498g at MCE (blue curve). As these accelerations are less than 0.35g, in accordance with TM 2.9.10 Geotechnical Analysis and Design R1 (May 22nd 2011) clause 6.10.13, additional earthquake pressures can be disregarded for the design of buried structures, provided that design for "at-rest" pressures is undertaken.

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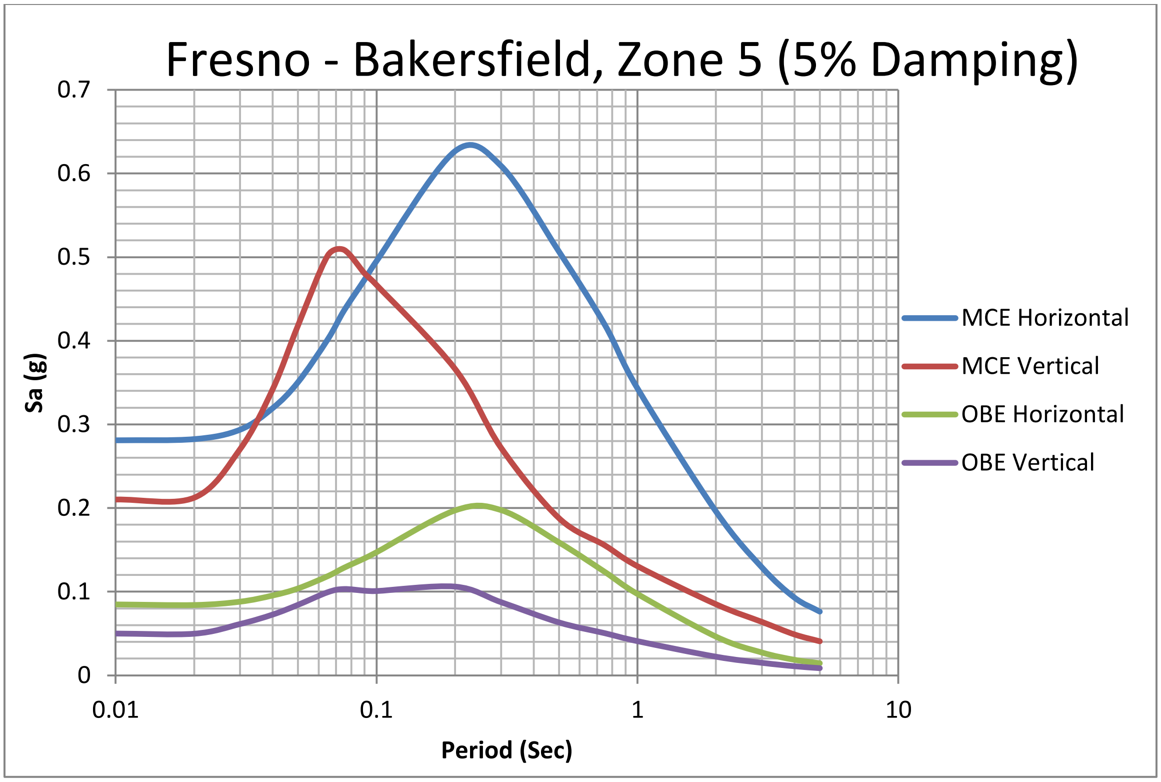

Figure 1.4-2
Design Response Spectra (Zone 5)

For Zone 5, the peak ground acceleration has been taken as the acceleration that corresponds to a period of 0.01 seconds — that is 0.0848 g at OBE (green curve) and 0.2810 g at MCE (blue curve). As these accelerations are less than 0.35g, in accordance with TM 2.9.10 Geotechnical Analysis and Design R1 (May 22nd 2011) clause 6.10.13, additional earthquake pressures can be disregarded for the design of buried structures, provided that design for "at-rest" pressures is undertaken.

1.4.3 Dynamic Performance

The representative structures have been analyzed to determine fundamental frequencies for primary modes of vibration. The frequencies have been checked for compliance with the requirements of TM 2.10.10, and it was found that there was no need to perform a dynamic structural analysis using actual high-speed trains for the preliminary design stage. A dynamic time history analysis has been performed in accordance with the track-structure interaction requirements of TM 2.10.10. Details of the analysis methodology are reported in the Seismic Design Plan, which is included as Appendix B.

1.4.4 Track Structure Interaction

Track/structure interaction analyses have been conducted for the selected typical structures to confirm feasibility of the structure form.

The structures analyzed demonstrated that the concepts are feasible, do not require rail expansion joints, do not require Zero Longitudinal Resistance (ZLR) track clips, and are capable of being developed to final design.

1.5 Geotechnical Assumptions Made for Preliminary Engineering

The recommendations of the Geotechnical Design Memorandum (GDM) included as Appendix A, have been followed, including:

- Soil parameters (γ_b, $\emptyset$, c_u).
- Assumed groundwater levels.
- The requirements of TM 2.3.2.

As no borehole investigation results are available for the FB Section, geotechnical advice was based on historic borehole records from California Department of Transportation (Caltrans) projects located in the vicinity of the route. The foundation spring stiffness used in the structural analysis has been based on a conservative interpretation of the soil parameters indicated by the borehole logs. The derivation of the stiffness matrices for the piled foundations are included with the *Geotechnical Data Report* (GDR).

1.6 Further Information Required to Develop the Design

It is expected that the design-build contractor will require more detailed factual information in order to address key design issues. The data required include but are not limited to the following:

- Borehole log details at each structure.
- Results of soils testing.
- Results of long-term monitoring of groundwater levels.
- Detailed knowledge of access routes and timing of access to site.
- Utilities information locations, depths, and risk.
- Details of working space adjacent to the BNSF for beam storage and site operations.
- Details of BNSF timetables for scheduling beam erection and other activities potentially impacting railroad operations.
- Construction constraints for working in floodplain areas and major river channels.
- Details of predicted regional subsidence.

1.7 Analysis Methodology

The assessment of HSR structures is concerned with the adequacy of the structural members, the serviceability of the structure, and its response to dynamic train loading. In addition to the structural members, rails must be assessed for their ability to withstand stresses and distortions to ensure that the design is compatible with the use of continuously welded rails, which is the preferred rail concept.

HSR structures must meet all of the following aspects of design:

- Dynamic behavior criteria.
- Track-structure interaction criteria.
- Rail serviceability criteria.
- Member capacity.
- Seismic performance.

The preliminary design of the structure encompasses all of these aspects, so that the worst possible effects are considered in each case. This has been achieved by creating multiple structural models using SAP and CSiBridge analysis programs, with each model configured with unique properties and load cases to capture the most onerous effects.

The performance of the structure and its acceptance is measured against the criteria given in Draft TM 2.10.10: Track-Structure Interaction R1 (February 29, 2012), American Association of State Highway and Transportation Officials (AASHTO) design codes, and Caltrans-specific design criteria. Structural loads are outlined in TM 2.3.2: Structure Design Loads R2 (April 20, 2011). Project-specific seismic design criteria are given in TM 2.10.4 Seismic Criteria R1 (May 26, 2011).

To envelope the worst cases, upper-bound and lower-bound model configurations have been utilized. The “soft” case considers a lower-bound model stiffness and upper-bound mass, whereas the “stiff” case considers an upper-bound model stiffness and lower-bound mass. These configurations are defined in TM 2.10.10 as Conditions 1 and 2, respectively.

1.7.1 Model Stiffness

The model stiffness is controlled through modification of the section properties. The soft model considers the cracked section properties of reinforced concrete (RC) members and nominal material properties. The stiff model considers gross section properties but takes expected material properties, rather than nominal.

Cracked section properties are derived from either the slopes shown in Figure 5.3 of the Caltrans Seismic Design Criteria (CSDC) or from a moment-curvature analysis. The moment-curvature relationship for RC sections is determined using the Section Designer module in SAP, by specifying an appropriate amount of reinforcement and axial load. Cracked properties are represented in the model by factoring the moment of inertia of the gross section properties, I_g. Typically the cracked stiffness ranges between 30% and 40% of the gross section stiffness; however, higher values can be justified with increased reinforcement.

The increased flexural stiffness due to expected material properties is represented in the model by factoring the properties of the gross section. The CSDC defines the expected compressive strength of concrete as 30% greater than the nominal strength, for concrete strengths of 4ksi or higher. For normal-weight concrete, this equates to an increase in the elastic modulus of 14%. In order to attribute cracked properties to flexural stiffness only, this factor is applied to the moment of inertia of the gross section, rather than the elastic modulus.

1.7.2 Model Mass

The mass that contributes to the modal and seismic analysis cases consists of the mass of the structural elements and the mass of the prospective train vehicle. The mass of footings and components below ground level have been omitted from the analysis.

Train loads are accompanied by an equivalent train mass, which acts at a distance of 8 feet above rail level. The definition of these loads has been configured so that they are attributed to the mass of the model only and are not included in any load cases.

For the purposes of upper-bound (stiff) and lower-bound (soft) model configurations, the total mass used in the analysis is controlled using the model mass definition function of SAP. The stiff model applies a 5% reduction to the total mass, whereas the soft model applies a 5% increase to the total mass.

A summary of model properties is as follows:

- Condition 1 (soft).
  - Lower-bound stiffness – uses cracked sectional properties for concrete members and nominal material properties.
  - Upper-bound mass – model/train mass increased by 5%.

- Condition 2 (stiff).
  - Upper-bound stiffness – uses gross sectional properties and expected material properties for concrete members.
  - Lower-bound mass – model/train mass decreased by 5%.

1.7.3 Boundary Conditions

In order to accurately represent the contribution of adjacent structures to the behavior of the HSR structure under consideration, boundary conditions are included in the model. For the case of structures that form part of a longer aerial viaduct, the approaching spans adjacent to the HSR structure are also modeled — typically 10 viaduct spans on either side.

For structures bounded by abutments and tracks on embankment, guidance is given in TM 2.10.10 regarding the required length of track extended away from the structure, which is dependent on the at-grade track type and properties of the rail fasteners.

Specific boundary conditions for each structure are discussed in their respective sections.

1.7.4 Foundation Properties

Foundations are modeled using pile springs, with the stiffness properties of each spring derived from historic ground investigation (GI) results and more recent GI results where possible. The stiffness matrix of the pile springs used in the models is derived using LPile, which simulates the performance of the soil/pile system, an approach further detailed in Appendix A of the GDM.

Group reduction factors have been applied to the pile stiffness matrix where necessary.

1.7.5 Rail Clip Properties

Rail clip properties vary depending upon the loaded or unloaded condition. In both cases the vertical stiffness is constant and equivalent to 4,000 kips per foot of track. The transverse stiffness is also constant and equivalent to 450 kips per foot of track. In the longitudinal direction, however, the rail clip stiffness is modeled as a bilinear coupling spring between the track and superstructure, the properties of which are shown in Figure 1.7-1. The rail clips are typically spaced at 27-inch centers, so the stiffness values mentioned are modified accordingly to give an equivalent stiffness. See TM 2.10.10 for further details.

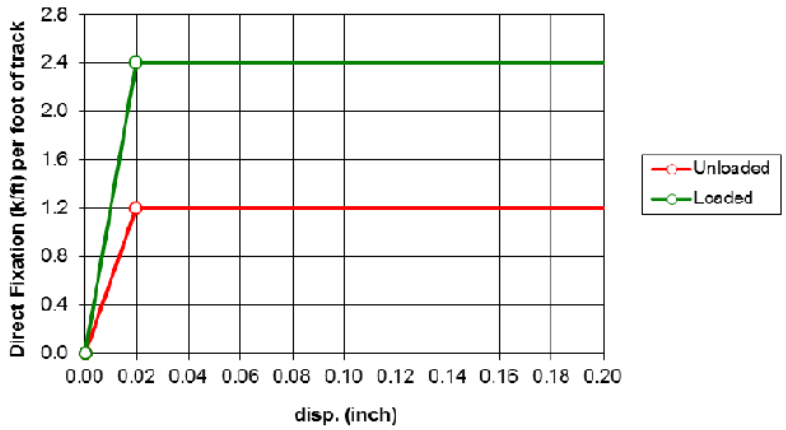

Figure 1.7-1
TM 2.10.10: Bilinear Coupling Spring Stiffness

Depending upon the position of the live load on the structure, the rail clip properties are modified to match those that are considered loaded — those beneath the train load — and those that remain unloaded.

At model boundaries, the horizontal boundary spring has stiffness and yield properties that represent the capacity of an infinite number of elastic fasteners. These stiffness and yield values are obtained from TM 2.10.10 and are dependent on the at-grade track type and rail fastener properties.

1.8 Dynamic Behavior

Frequency analysis is used to ensure that the structure is proportioned to resist resonance effects and that the code-prescribed values relating to the dynamic amplification of loads remain applicable. The three specific frequencies of interest are the vertical, torsional, and transverse frequencies.

Vertical and torsional frequencies are investigated on both the soft and stiff models. TM 2.10.10 gives limits to these frequencies for both the upper-bound and lower-bound cases, for which the limits are dependent on the span lengths and the support conditions. Transverse frequencies are evaluated only with the soft configuration, with the added condition that the structure is fully fixed at the bearings so that only the flexibility of the superstructure is considered.

1.9 Track/Structure Interaction and Rail Serviceability

The structure is required to meet both track structure interaction and rail serviceability criteria in order to ensure that structural deformations and adverse dynamic effects due to a moving train load are limited. These effects include excessive rail stresses, excessive structure deformations, the risk of train derailment due to relative twisting or misalignment of the rails, rail break, excessive rail or track wear, and poor track maintenance.

Several load permutations are evaluated, with each case having specific limits of acceptance. These cases are defined as groups, and they vary by the number of loaded tracks, the consideration of traction, braking and centrifugal forces, thermal effects, and the occurrence of a seismic event. Note that only OBEs, as defined in TM 2.10.4, are considered for the assessment of track-structure interaction and rail serviceability.

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The following is a summary of the load cases evaluated for the track-structure interaction (Groups 1–3) and rail serviceability (Groups 4 and 5):

- Group 1a: $(LLRM + I)_{1.}$
- Group 1b: $(LLRM + I)_2 + CF_{2.}$
- Group 1c: $(LLRM + I)_m + CF_{m.}$
- Group 2: $(LLRM + I)_1 + CF_{1.}$
- Group 3: $(LLRM + I)_1 + CF_1 + OBE$.
- Group 4: $(LLRM + I)_2 + LF_2 \pm T_{D.}$
- Group 5: $(LLRM + I)_1 + LF_1 \pm 0.5T_D + OBE$.

Where,

LLRM = Live Load from Modified Cooper E-50 train

LLRR = Maintenance and construction train load (Cooper E-50)$(LLRM + I)_1$ = one track of LLRM plus impact

$(LLRM + I)_2$ = two tracks of LLRM plus impact

$(LLRM + I)_m$ = multiple tracks per TM 2.10.10 Section 6.9.3 of LLRM plus impact

LLRM = modified LLRR live load

I = vertical impact factor

CF_1 = centrifugal force (one track)

CF_2 = centrifugal force (two tracks)

OBE = operating basis earthquake

LF_1 = braking forces

LF_1 = braking and traction forces

T_D = temperature differential

Note that water loads are applicable but have been omitted from these definitions as level information is unavailable.

The models are identical, with the exception of the rail clip stiffness assignments (see Section 1.7.5), the location of the train mass assignments, and the location of the applied live/braking/traction loads.

All structures have been sized to comply with the frequency limits of TM 2.10.10. It has therefore not been necessary to carry out a dynamic analysis with actual high-speed trains to demonstrate compliance with vertical deck acceleration limits.

Note that trainset details, including the mass, stiffness, and damping characteristics of the trainsets, have not been made available at the time of preliminary design. Passenger comfort analyses have therefore not been investigated.

1.9.1 Displacement Model Configuration

As maximum displacements present the worst case, the soft configuration — lower-bound stiffness and upper-bound mass — is of most interest.

In the analysis, train locations have been chosen to envelope the most onerous effects on the structure. For the case of rail serviceability, the locations selected are those that will develop the highest displacements and rotations at the structure expansion joints and thus produce the worst cases, both in terms of deflection and rail stresses. As such, several models are required, each one denoted by either the specific live load location (Position A, B, etc.) or the expansion joint under consideration (Abutment 1 or Bent 11, for example). See Figure 1.9-1 for a typical displacement model live-load layout.

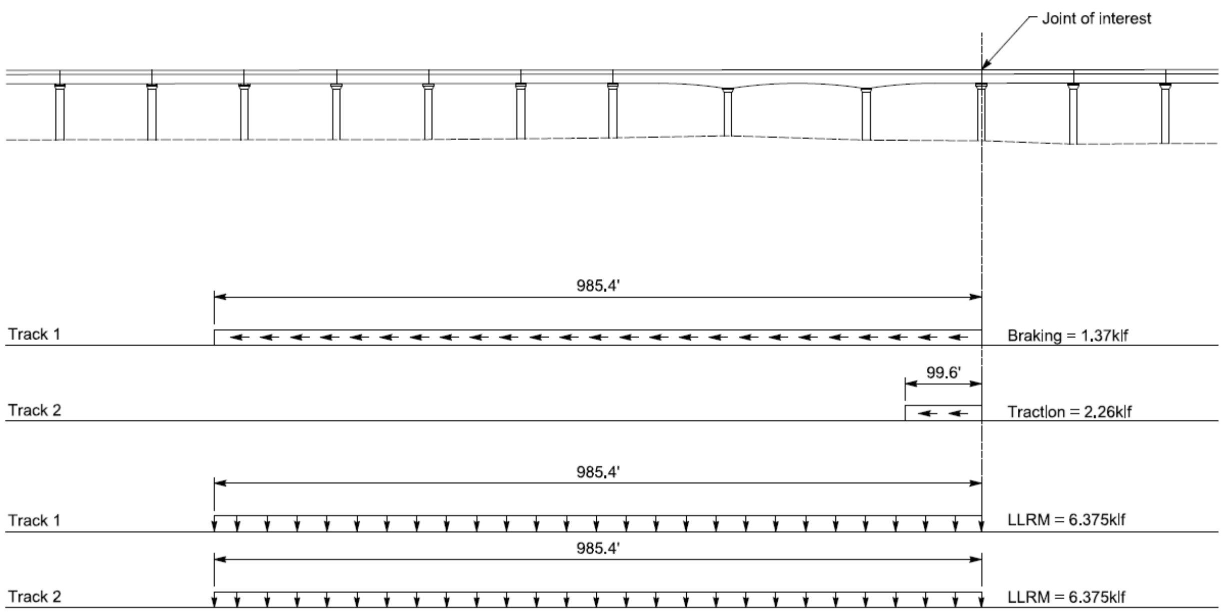

Figure 1.9-1
Typical Displacement Model Live-Load Layout

1.10 Member Capacity

The Regional Consultant has assessed the structural adequacy of each HSR structure against the requirements of AASHTO (with Caltrans amendments) in addition to the CSDC. For the purpose of preliminary design, only the key structural members have been checked; ancillary components such as shear keys and connections have not been reviewed in any detail, and it is expected that these will be verified at the detailed design stage. In the case of the crossover structure, however, the integral connection between the column/edge beam/joist has been checked to verify the adequacy of load transfer in the joint during a seismic event.

The key members checked are as follows:

- Superstructure.
  - Steel truss members and beams (SAP steel section designer).
  - RC beams (SAP concrete section designer and/or hand calculations).

  - RC deck slab (SAP concrete designer for 2-D shell elements and/or hand calculations).

- Substructure.

  - RC columns (SAP concrete section designer).

- Foundations.

  - RC pile caps (hand calculations).
  - RC piles (hand calculations).

The in-built section designer functions of SAP and CSiBridge check the elements for adequacy based upon preselected load cases. The primary load cases of interest are Strength 1, which accounts for ultimate temporary loads (live and thermal), and Strength 5, which is the OBE seismic case as defined in TM 2.3.2:

- Strength 1: 1.25/0.9DC + 1.75$(LLRR+IM)_2$ + 1.2/0.5TU.

- Strength 5: 1.25/0.9DC + 0.5$(LLRR+IM)_1$ + 1.1OBE.

Where,

DC = dead load of structural components and permanent attachments

LLRR = Maintenance and construction train load (Cooper E-50)$(LLRM + IM)_1$ = one track of LLRM plus impact

$(LLRM + IM)_2$ = two tracks of LLRM plus impact

IM = vertical impact factor

TU = uniform temperature effects

OBE = Operating Basis Earthquake

1.10.1 Ductile and Capacity Protected Components

MCE events, accounted for in the Extreme 3 load case, are not directly considered as part of the member capacity check. Plastic hinging mechanisms occurring in the columns will protect the adjacent components from excessive loading during an MCE. These components are therefore designed for the potential overstrength moments and forces that may be transferred due to the plastic hinging, where the overstrength moment is defined as the plastic moment capacity of the hinging member multiplied by 1.2. An associated overstrength shear force is also considered, taken as the overstrength moment divided by the height of the column, or in the case of plastic hinges being located at both the base and top of the column, the moment divided by half of the column height.

The plastic moment of the column is calculated using a moment curvature (M-ϕ) analysis with the Caltrans idealized curve, as defined in Section 3.3.1 of the CSDC. The axial load used to derive the M-ϕ relationship is taken as the nominal axial load in the column due to dead and superimposed dead loads, and overturning effect for multi-column bent.

Columns with plastic hinges are assessed both for elastic capacity in the force models, as well as inelastic capacity and ductility with pushover models (see Section 1.4.2).

1.10.2 Force Model Configuration

Unlike the track structure interaction models, where maximum displacements are desired, the approach with the capacity check is to configure the models so as to maximize the forces in the members. The modeling properties are therefore similar to the stiff models used during the track-structure interaction assessment, on the basis that stiffer elements experience increased force demands. Rather than specify an upper-bound mass in the force models, however, only a nominal mass has been taken.

Train locations have been selected to generate the highest member forces. From the perspective of the columns, the most onerous load configuration is that which exerts the most horizontal force at the top of the column, thus exerting the highest shear forces and moments.

By comparison and considering the load combinations given in TM 2.3.2, the forces imposed in the longitudinal direction due to traction and braking are the most onerous. A staggered train arrangement is therefore adopted, whereby the one train in braking is centered over the column/bent of interest and the other train with traction is located so that the traction portion is situated over the column/bent of interest (see Figure 1.10-1). Analysis results of Strength 1 and Strength 5 are reported in individual sections in this report.

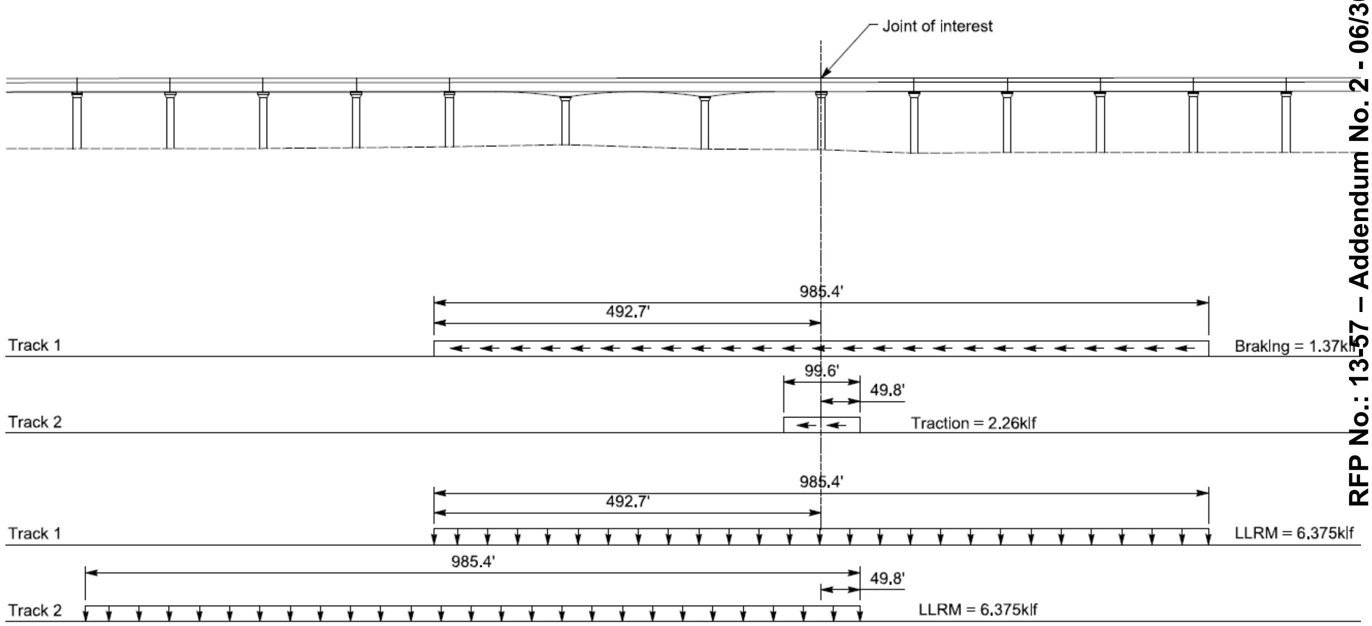

Figure 1.10-1
Typical Force Model Live-Load Layout

1.10.3 Redundancy Check

In the case of truss structures, an additional check is undertaken to evaluate the structures' performance with one of the diagonal members removed. This is a redundancy check with a requirement that the structure will not collapse under this condition. The in-built section designer module of SAP and CSiBridge was used; however, this considers only elastic capacity, which is conservative. Under the "no-collapse" performance requirement under this check, consideration for plastic capacity of the members could be warranted — this may be investigated further during detailed design.

1.11 Seismic Performance

The seismic performance of the structure is outlined in TM 2.10.4 and also the CSDC. During an OBE event, the structure is required to remain elastic, and this check is made during the member capacity check in the force models. During an MCE event, a no-collapse condition is required, which requires an assessment of the hinging mechanisms and ductility of the structure.

Plastic hinges are assigned to certain locations of the structure in order to control the method of structural failure and also to limit the forces/moments transferred to components where it is desirable to have them capacity-protected, e.g., footings. In RC columns, plastic hinges are located at the base, with additional hinges placed at the tops of the columns in the cases of the crossover structures. Hinge lengths and properties are determined in accordance with the CSDC.

To satisfy the OBE seismic performance criteria, it must be shown that the plastic hinges will not develop during an OBE event, which is demonstrated during the member capacity check by ensuring that the columns remain elastic.

During an MCE event, the hinges develop and become inelastic, and their behavior is then assessed using global and local pushover analyses in SAP. The two criteria of interest are specified in the CSDC as the Displacement Ductility Demand and the Displacement Ductility Capacity, which are the global and local member checks respectively.

Nonlinear analysis has been performed as checking balanced stiffness and balanced frame geometry per CSDC Section 7 is not required.

1.11.1 Displacement Ductility Demand

The Displacement Ductility Demand is the measure of the member ductility after hinging has occurred. It is given by the following relationship:

$$\mu_D = \Delta_D / \Delta_{Y(i)}$$

Where,

Δ_D = global displacement

$\Delta_{Y(i)}$ = yield displacement

The global displacement, Δ_D, is the displacement of the column/bent during an MCE event — taken at the tops of the columns — and is determined by applying the MCE time history cases to the full structural model in SAP or CSiBridge. Hinge properties are assigned to the columns in the desired locations so that inelastic displacements are also taken into account. To conservatively take the maximum displacements under MCE loading, the soft model configuration was used; however, train loads/masses were not considered.

The yield displacement, $\Delta_{Y(i)}$, is determined from a pushover analysis in both the longitudinal and transverse directions. The analysis applies accelerations to the model in progressive steps and displays the status of the hinge at each step. At the point that the hinge yields, the displacement at the top of the column is recorded and taken as $\Delta_{Y(i)}$. To conservatively take the minimum displacements that relate to column hinging, the stiff model configuration was used.

In the crossover structures, there are many columns that are seen to yield at different pushover steps. In this case, the pushover step that presents the first column yield is taken as the reference and only those columns that yield during this step are reviewed for displacement ductility.

The target displacement ductility demand varies depending on the support condition and fixity, but is <4 and <5 for single- and multicolumn bents, respectively. Displacement ductility less than 1 indicates that the column remains elastic during the MCE event.

1.11.2 Displacement Ductility Capacity

The displacement ductility capacity is a local check of the column ductility irrespective of foundation and superstructure flexibility. It is given by the following relationship:

$$\mu_C = {\Delta_C}/{\Delta_Y^{col}}$$

Where,

Δ_C = collapse displacement

Δ_Y^{col} = yield displacement

The pushover analysis is undertaken on a local model of the column bent with fixities assigned to represent the assumed fixities of the superstructure and foundations. Typically the foundations are assumed to be fully fixed, so the bases of the columns in the local model are rigid. The fixities at the top of the column vary depending upon the expected behavior and will typically be free if the column is assumed to act as a cantilever or rotationally fixed if the column is part of a frame or multicolumn bent. No translational restraints are applied to the tops of the columns.

The pushover analysis is conducted in both the longitudinal and transverse directions, and in each case the displacements of the columns are recorded for the initial yield and then collapse stages of the hinges. Displacement ductility capacity is discussed on a case-by-case basis in other sections.

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Section 2.0

Conejo Crossover Structure

2.0 Conejo Crossover Structure

The Conejo Viaduct is 5,049 feet in length and composed of three sections: the E Conejo Avenue standard viaduct, the BNSF crossing, and the S Peach Avenue standard viaduct. The BNSF crossing structure is considered to be nonstandard and complex, and is the subject of this analysis.

For this analysis, the BNSF crossing portion was originally conceived as a 950-foot-long elevated slab, supported on multiple columns to either side of the BNSF railroad corridor. The 6-foot-diameter crossover columns are positioned at 30-foot centers along the length of the structure and are founded on a single 9-foot-diameter drilled shaft pile of 170-foot depth. Piers were later moved outside the BNSF ROW requiring an increase in the length and span of the structure, which is now 1,429 feet in length. To confirm that the concept of the structure at the new length was still valid, a submodel consisting of the end panel of the extended structure was analyzed to confirm that frequencies and seismic performance were similar to the original model. This analysis showed that remaining within the lower frequency boundary is necessary to stiffen the span of the structure by adding a second line of columns on the BNSF right-of-way boundary. This modification of the structure to increase the end stiffness confirms that the original analysis model can be regarded as conservative.

The slab section is constructed from 6-foot-deep, precast, PC beams and supported on 12-foot-deep by 30-foot span in situ concrete column cap beams, which run parallel to the railway. The beams span approximately perpendicular to the BNSF tracks and are placed immediately adjacent to one-another; typically this gives a spacing of 4 feet on centers. The deck slab is 6 inches in thickness and is intended to act compositely with the beams. The superstructure has been divided into individual thermal units of approximately 150- to 200-foot length to reduce the thermal displacement and force effects. Movement between adjacent thermal units of the slab is controlled by dowelled connections, which allow relative longitudinal and vertical displacements but not relative transverse displacement. A similar dowelled connection is provided between the end panel of the slab and the adjacent span of the standard viaduct.

The standard spans of the viaduct are formed from precast, prestressed box girders and seated on RC columns, which are in turn supported on a pile cap with a group of 4no. 6-foot-6-inch-diameter drilled shaft piles. Due to clearance constraints near to the BNSF right-of-way and reduced loading, the columns immediately adjacent to the crossover structure modify the general foundation arrangement by using a two-pile group with a narrower pile cap.

2.1 Structure Importance Classification

TM 2.3.2 paragraph 2.2.1 defines all structures supporting the high-speed tracks to be primary structures because they will be required to be reinstated to allow resumption of train service after an earthquake. This classification implies the following:

- Design life is 100 years.
- Seismic design must comply with TM 2.10.4.
- When applying the AASHTO LRFD code, values for the importance (hI), ductility (hD), and redundancy (hR) factors have been chosen as follows:
  - hI = 1.05.
  - hD = 1.05 for strength calculations.
  - h_R = 1.05 for nonredundant elements, 1.0 otherwise.

2.2 Key Design Features and Site Constraints

2.2.1 Dowel Connections

Dowel connections are located at the breaks between adjacent thermal units of the deck slab and at the interface connections between the crossover structure and the standard viaduct sections. The purpose of the dowels is to control the relative movement between the thermal units and, in particular, the movement at the rails. The dowels are aligned to be parallel with the rail axes at the interface between the units to ensure that the relative structure movement is also along the rail axis. This ensures that lateral distortions are minimized. The dowels are assumed to allow relative rotation about the transverse axis and displacement in the longitudinal and vertical directions, but they limit all other degrees of freedom.

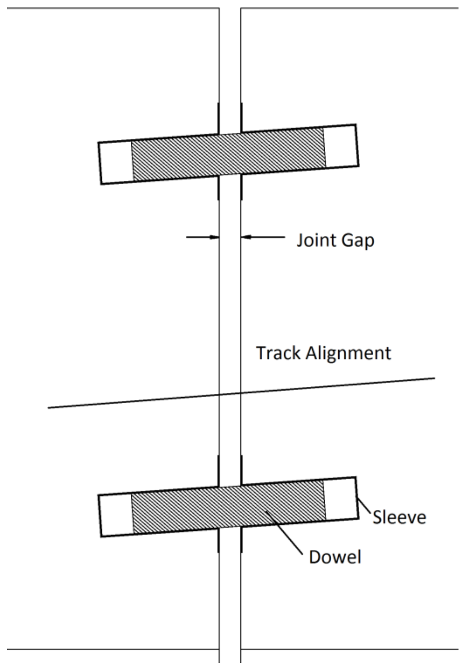

Figure 2.2-1
Detail of Dowels in One-Column Cap Beam

As the dowels are aligned with the rails, expansion joints between the adjacent thermal units are not required to be perpendicular to the rail and are not in this case. It has instead been assumed that the joints will be aligned parallel to the cross beams. This requires the joint design to consider a minor component of lateral displacement with longitudinal displacement, but this is considered to be within the capability of typically available structure joints. Alternatively, for the simplification of track clip arrangement the joint could be made to be perpendicular to the rail in the vicinity of the HSR tracks and revert to being parallel to the beams outside this area.

The merits of these variations should be investigated further during the design development stage.

2.2.2 Ground Conditions

The geotechnical parameters used for the analysis are based on historic borehole records from Caltrans projects located in the vicinity of the route, as no project-specific or local borehole data were available. The foundation spring stiffness has therefore been based upon the lower-bound

interpretation of the soil parameters, using the nearest borehole data and engineering judgment. Detailed design will be based on investigation results, which are expected to demonstrate that this approach is conservative.

See the GDR in Appendix A for details of parameters and spring stiffness used in the analysis.

2.2.3 BNSF Future Provision

Double tracking is planned by the BNSF for several locations between Port Chicago and Bakersfield. It is understood that the BNSF has no plans to install additional tracks in locations where double tracking is already provided. The Conejo Viaduct spans over two existing BNSF tracks, so no provision for future tracks has been considered necessary. The geometry of the Conejo crossover structure has therefore been established on the basis of two BNSF tracks only.

2.3 Summary of Analysis and Results

The PC box girder spans on either side of the BNSF crossover structure are classified as standard structures and do not fall within the scope of this preliminary design. They have been modeled, where necessary, in accordance with the seismic design criteria to ensure that the behavior of the BNSF crossover structure is fully representative.

All sections have been checked for resonance effects, rail serviceability and track-structure interaction limits, and force demands. In all cases the structure has been found to be satisfactory.

Based upon the calculations thus far, it appears that the preliminary designs are in full compliance with the TMs and are capable of being developed into a fully compliant design solution. Refer to the *Package 2-3 Structures Calculations Report* for the complete analysis and results.

The main results are summarized in Tables 2.3-1 to 2.2-15.

2.3.1 Modeling

Both SAP and CSiBridge modeling programs were used for the analysis of the Conejo Viaduct. Several models of each section were required in order to represent the different conditions of the structure for different loading cases and for different design checks, in accordance with TM 2.10.4 and TM 2.10.10.

The structural columns, cross beams, rails, and RC girders were represented by stick elements. Piles were represented by nonlinear springs, using equivalent stiffness values to correctly model the soil structure interaction based upon soil parameters in Appendix A. The pile cap and pile group effects were modeled using rigid links connecting the top of the piles to the column elements. All standard viaduct spans were connected to the bent cap elements with linear bearing springs, with the bridge articulation represented by either pinned or rolling spring properties. In the case of the transverse frequency analysis, pinned restraints were added in place of the bearings, as only the flexibility of the superstructure is to be considered. Note that for the type of structure under consideration, the fixity requirement of TM 2.10.10 fully restrains the superstructure from all transverse movement.

Foundation arrangements for the standard spans were those used in the Authority's representative's design for the standard viaducts and have been used accordingly in the structural models. These foundations have been checked using LPILE and Pilset, and have been found to have adequate capacity.

Linear and nonlinear springs were used to represent boundary conditions and stiffness in the model. Nonlinear boundary springs were used to model the nonlinear behavior of rail clips and pile foundations. However, when running linear analyses such as model analysis and response spectrum analysis, these springs are assumed to operate in the linear stiffness range and are therefore modeled as linear boundary springs. In accordance with TM 2.10.10, upper and lower bound stiffness were taken into account as were upper and lower bound mass.

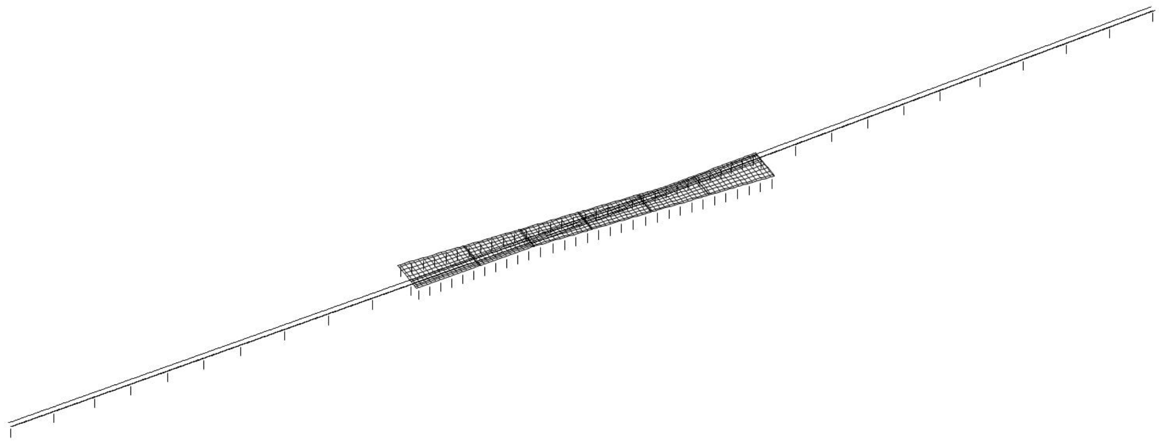

Figure 2.3-1
SAP Model

2.3.2 Frequency Results

The vertical, torsional, and transverse frequencies of the structure were evaluated to ensure that they meet the required dynamic criteria, as defined by TM 2.10.10. In the case of the vertical and torsional frequencies, two conditions were assessed: the first with upper bound mass and lower bound stiffness (Condition 1), the second with lower bound mass and upper bound stiffness (Condition 2). Condition 1 was also adopted for the transverse frequency analysis, as it generated the most onerous frequencies in this case.

In all thermal units of the crossover structure, the natural frequencies were found to be within the defined limits. The thermal unit with a transverse span of 74.2 feet and a column height of 35.5 feet produced the highest lower bound frequency limit with an effective span length of 62.9 feet. Whereas the unit with a transverse span of 100.4 feet and column height of 35.5 feet produced the lowest upper bound frequency limit with an effective span length of 74.3 feet. Although the original SAP model has transverse spans up to 100.4 feet, the additional 115-foot transverse spans model is checked for vertical frequency. See Table 2.3-1 for an envelope summary of the limits and most onerous natural frequencies for all thermal spans. For span specific limits and natural frequencies, see the structural calculations. No torsional frequencies were found below 4.36 Hz for condition 1 and 5.22 Hz for condition 2.

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Table 2.3-1
Conejo Crossover Structure Frequency Check Result Envelope

	Vertical Frequency L=100 ft (Hz)	Vertical Frequency L=115 ft (Hz)	Torsional Frequency L=100 ft (Hz)	Torsional Frequency L=115 ft (Hz)	Transverse Frequency (Hz)
Lower Limit	3.73	3.54	5.36 condition 1 6.33 condition 2	4.36condition 1 5.22 condition 2	1.2
Upper Limit	9.27	8.64	N/A	N/A	N/A
Condition 1	4.46	3.63	greater than 5.36	greater than 4.36	1.54
Condition 2	5.27	4.354	greater than 6.33	greater than 5.22	N/A

The fundamental frequency in the vertical direction is first observed in the modal results at the ends of the crossover structures, where the spans are at their longest due to the tapered geometry in plan. It has been found that the frequency in this direction is sensitive to the stiffness provided by the cross-beams and column sections, but also that it is particularly sensitive to the vertical stiffness of the foundations. Due to the soft soil case that has been considered in the design, the frequencies found are therefore also conservative.

It has been found that vertical frequency requirements govern the section dimensions of the crossover structure, with deeper sections needed to provide sufficient stiffness in order to satisfy TM 2.10.10. The section sizes specified are therefore larger than would be required from consideration of other effects such as strength.

As the vertical frequencies are governed in large part by the ground conditions, access to site-specific GI data may reveal more beneficial soil parameters and permit savings with the refinement of the design. This can be investigated in further development of the design.

2.3.3 Rail Serviceability and Track-Structure Interaction Results

The crossover structure was analyzed for deflections and rail stresses, and evaluated against the limits prescribed in TM 2.10.10. This included the assessment of global deflections of the structure, the relative rotations and displacements at the rails and expansion joints, and the relative twist of the deck.

Several SAP models were developed to model train loads at the most onerous locations on or immediately adjacent to the crossover structure. The various train locations were coupled with the load permutations and cases specified in TM 2.10.10 to envelope the worst deflections in the structure.

It has been demonstrated from the analysis that the Conejo Crossover Structure meets all of the requirements of TM 2.10.10 and that all rotations, deflections, and rail stresses are with limits. See Tables 2.3-2 to 2.3-8 for a summary of the results. Results shown are for the worst cases only. Joint-specific results can be found in the complete calculation report along with supporting calculations.

Table 2.3-2
Conejo Crossover Structure Track Serviceability Results (1)

Group	Vertical Deflection (in) L = 100 ft		Transverse Deflection (in) L = 286 ft	
	Limit	Conejo	Limit	Conejo
Group 1a	0.343	0.169	1.135	0.165
Group 1b	0.500	0.231	2.195	0.178
Group 3	N/A	N/A	3.546	0.503

Table 2.3-3
Conejo Crossover Structure Track Serviceability Results (2)

Group	Rotation about Vertical Axis (rad)		Rotation about Transverse Axis (rad)	
	Limit	Conejo	Limit	Conejo
Group 1a	0.0007	0.00017	0.0012	0.0008
Group 1b	0.0010	0.00003	0.0017	0.0013
Group 2	0.0021	0.00017	0.0026	0.0008
Group 3	0.0021	0.00057	0.0026	0.0010

Table 2.3-4
Conejo Crossover Structure Track Serviceability Results (3)

Group	Deck Twist (rads/10ft)	
	Limit	Conejo
Group 1a	0.0011	0.00104
Group 1b	0.0011	0.00009
Group 2	0.003	0.00110
Group 3	0.003	0.00121

Table 2.3-5
Conejo Crossover Structure Track Structure Interaction Results (1)

Group	Relative Longitudinal Displacement	
	Limit	Conejo
Group 4	1.806	1.543
Group 5	2.733	1.495

Table 2.3-6
Conejo Crossover Structure Track Structure Interaction Results (2)

Group	Relative Vertical Displacement	
	Limit	Conejo
Group 4	0.25	0.086
Group 5	0.50	0.071

Table 2.3-7
Conejo Crossover Structure Track Structure Interaction Results (3)

Group	Relative Transverse Displacement	
	Limit	Conejo
Group 4	0.08	0.019
Group 5	0.16	0.020

Table 2.3-8
Conejo Crossover Structure Track Structure Interaction Results (4)

Group	Permissible Axial Rail Stress	
	Limit	Conejo
Group 4	±14	12.0 to -8.6
Group 5	±23	22.6 to -18.7

2.3.4 Force Results

The key components of the crossover structure have been checked for structural adequacy to assess the validity of the section sizing. In addition to the crossover structure, sections of the typical viaduct that interface with the crossover were also checked. This included the box girder spans immediately adjacent to the crossover and the columns supporting these spans. The force checks comprised the RC design of the columns and pile caps, feasibility of the post-tensioned cross beams at the specified sizes, and the RC design of the piled foundation options.

Table 2.3-9
Column Strength Check – Load Case, Axial, and Flexural at Governing Locations

	Viaduct Column	
Load Combination	**Strength 1**	**Strength 5**
Axial Demand (k)	1,317	1,185
Moment M3 Demand (k-in)	141,369	56,066
Moment M2 Demand (k-in)	115,814	46,586
Demand/Capacity Ratio	0.911	0.468

Table 2.3-10
Pile Strength Check – Governing Axial/Moment Load Interaction per Pile

	Strength 5
Governing Axial Demand (k)	984
Flexural Demand (k-in)	165,231
Moment Demand/Capacity Ratio	0.853

2.3.4.1 Dowel Forces

The dowel elements have been modeled as nominal 12-inch-diameter steel pins, with 2 dowels placed at each joint on the crossover. The intermediate joints on the crossover are much broader than those between the crossover and typical viaduct, which would permit a greater number of dowel elements to be installed; reducing dowel stresses and diameters. The merits of a greater number of dowels can be evaluated in further design developments. For consistency, a 2 dowel configuration has been maintained in all joint locations in the structure.

The forces in the dowels have been determined and compared with the capacity of the 12-inch-diameter steel sections. In all load cases and configurations, the strength of the dowels has been found to be satisfactory. See Table 2.3-11 for a summary of the results.

Table 2.3-11
Conejo Crossover Structure Dowel Capacity Results

Load Case	Shear Force, V3		Bending Moment, M2	
	Capacity, V_r	Conejo	Capacity, M_r	Conejo
Strength 1	2620	727	6780	3757
Strength 5	2620	258	6780	1207

2.3.4.2 End-Span Check

The adoption of dowel connections at the joint between the crossover structure and the typical viaduct results in forces being transferred from the crossover into the adjacent viaduct spans (end-spans). These spans have therefore been checked for structural adequacy as part of the overall viaduct assessment.

The main variation between the end-spans and the standard spans is the torsional force that is induced in the box section due to the transverse load transfer from the dowels. The effects of the connection on the moments about the minor axis of the box section were also evaluated.

The check was conducted as a comparison between the shear stresses observed in the box section webs from the standard 120-foot-span sections, and the stresses in the Conejo Crossover Structure's end-span section. The shear stresses were derived from the applied shear and torsion forces, determined using the Conejo SAP models. The stresses in a 120-foot standard span were taken from the boundary spans in the Conejo Crossover Structure models.

The comparison shows that the forces transferred from the crossover increase the shear stresses in the webs of the end-span box girder by 20%. This is a manageable increase that can be accounted for by a modification of the box girder shear reinforcement. In the top and bottom flanges, the maximum shear stress increases by 65%, but it should be noted that the stresses in the flanges of the typical sections are initially small, and so stresses in the end-span sections are similarly small.

The design moments about the minor axis of the standard box girder are shown to decrease in the end-span section. This is attributed to the reduced fixity of the end-span provided by the column immediately adjacent to the crossover structure. As the bearings for only the one span are located over the centroid of the end column, in comparison with a typical intermediate column that is loaded eccentrically and by two spans, there is less resistance to the twisting of the column. When this is considered in terms of the transverse plane of bending, the end-column represents a pinned support in comparison to the rigid typical column supports. The maximum transverse moments in the column are therefore observed to be 25% less than those of the typical viaduct sections.

2.3.4.3 Thermal Load Effects

In developing the structure model it was initially thought that a 300-foot spacing of joints between panels of the structure slab would be satisfactory as this was close to the maximum thermal length requirements of TM 2.10.10. Initial test analysis runs showed that, contrary to the established design philosophy of the CSDC, seismic-induced loading would not be the primary driver of the design for these structures. These analyses showed that the design was primarily governed by both the frequency requirements of TM 2.10.10 and thermal loads when applied in the Strength 1 load combination.

Thermal loading was particularly dominant due to the rigid restraints provided by the columns to the superstructure, resulting in large forces being transferred from the column cap into the columns. This restraint also had the effect of constraining the thermal expansion of the superstructure at the ends of each respective thermal unit, resulting in downward hogging during thermal expansion and uplift during contraction.

To moderate these forces the model of the structure was revised to incorporate more frequent joints, typically at 150- to 180-foot spacing. This had the effect of substantially reducing the thermal effects. It is still possible that thermal loads may be the governing design case, though it is more likely that the seismic case will govern the design. Where thermal forces govern the design it may be prudent for the detailed designers to consider refining the joint spacing to further reduce the thermal forces.

The dowel connections between panels were also susceptible to high loads in the Strength 1 combination. The constraints provided by the dowels have the potential to restrict the natural thermal movement of the structure both transversely and vertically, due to the uplift/hogging effects. For this reason the dowels have been articulated to allow vertical displacement and rotation about the transverse axis in an effort to reduce the forces imposed in the elements while retaining the benefit of lateral restraint from having the dowels. The increase in numbers of joints as described above also led to a substantial reduction in dowel forces. Relative displacements and rotations between adjacent thermal units were found to be within limits with this configuration.

2.3.5 Seismic Results

2.3.5.1 Columns

The displacement capacity and displacement ductility demand of the columns were assessed in accordance with the CSDC. Each column has a 1 % reinforcement ratio. The reinforcing steel detail is shown in the following figure. The assessment was completed using a combination of global and local SAP models and running pushover analyses in both the X and Y directions.

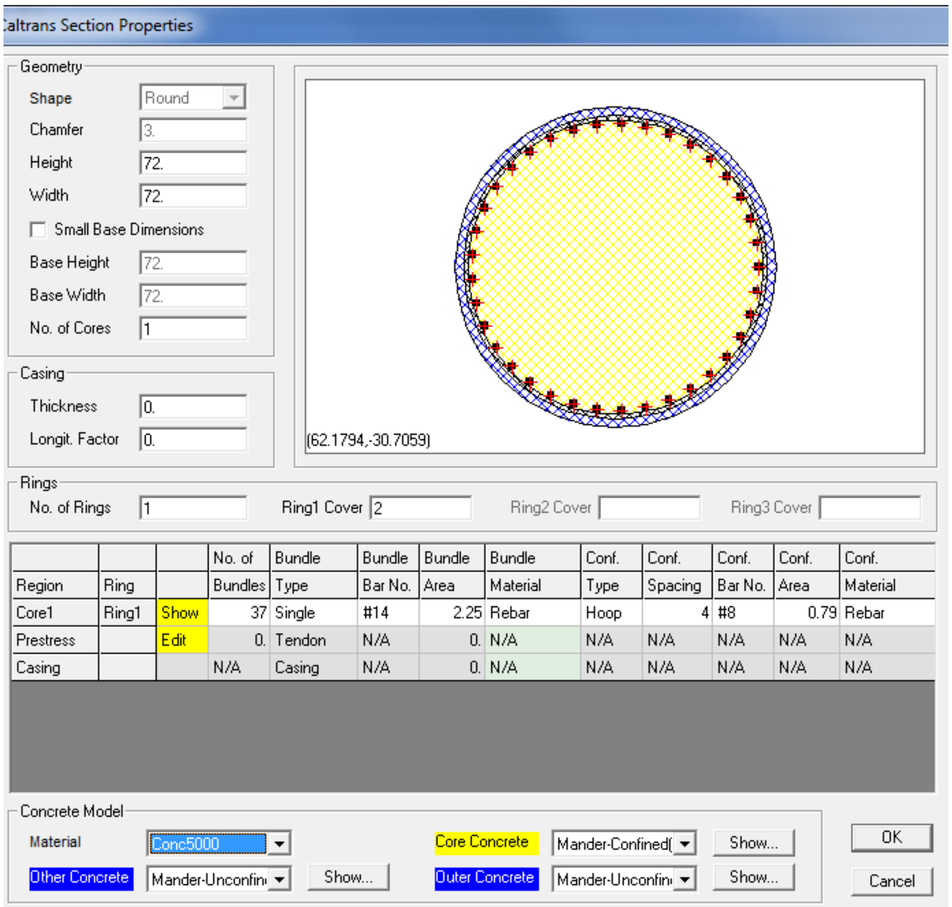

Figure 2.3-2
Section Properties – Conejo Crossover Structure 6-Foot Diameter Column

Due to the number of columns, pushover analysis proved to be difficult, as the software struggled to record the exact yield and collapse states for each and every column. Columns are seen to yield at different pushover steps. In displacement ductility demand check, the pushover step that presents the first column yield is taken as the reference and only those columns that yield during this step are reviewed for displacement ductility. In the case of the ductility capacity check, this was not an issue as only a local model of a single column was required. It has been demonstrated from the analysis and calculations that the Conejo Crossover Structure is structurally viable and meets the requirements of the CSDC. See Tables 2.3-9 to 2.3-10 for a summary of the results.

Table 2.3-12
Seismic Displacement Check – Displacement Demand Ductility Check

	Displacement Ductility Upper Limit	MCE Displacement ΔD (in)	Yield Displacement ΔY (in)	Displacement Ductility, μD
Longitudinal	5	0.340	1.357	0.251
Transverse	5	1.011	1.713	0.590

Table 2.3-13
Seismic Displacement Check – Capacity Ductility Check

	Capacity Ductility Lower Limit	Yield Column Displacement ΔYCOL (in)	Collapse Column Displacement ΔcCOL (in)	Capacity Ductility, μC
Longitudinal	3	0.102	0.642	6.27
Transverse	3	0.099	0.642	6.48

Table 2.3-14
Seismic Displacement Check – Displacement Demand/Capacity Ratio Check

	MCE Displacement Demand ΔD (in)	Displacement Capacity Δc (in)	Demand/Capacity
Longitudinal	0.340	1.999	0.17
Transverse	1.011	2.355	0.43

Table 2.3-15
Column Strength Check

	Envelope
Plastic Moment, Mp (kip-in)	182,261
Overstrength Moment, Mo (k-in)	218,713
Overstrength Shear Demand, V (kips)	1,037
Shear Capacity ØVn (kips)	2,280

2.3.5.2 Foundations

The demand forces to be used for the foundation design are specified in the CSDC and include the service level moments, shears, axial loads and the moment demand induced by the column plastic hinging mechanism. The moment derived from the plastic hinging mechanism is taken as the plastic moment of the column, multiplied by an overstrength factor of 1.2 to give the overstrength moment demand. The adoption of the overstrength factor is based upon the seismic design philosophy whereby the columns will always yield at the MCE event to protect the pile. The factor is required to account for risk that the column may develop a greater plastic moment capacity than the idealized values used in the design. An axial-moment diagram from the section designer module in CSiBridge was used to check the reinforcement design of the pile. A minimal reinforcement ratio of 1% and #8 at 4-inch tie spacing was used for 9-ft diameter pile. Pile shear and moment demand derived from column overstrength moment are checked against capacity.

See Table 2.3-16 for a summary of the results.

Table 2.3-16
Pile Strength Check – Column Overstrength Demand per Monopile

	Max Axial	Min Axial
Extreme 3 Axial Demand	1,037	-935
Overstrengh Design Shear Demand (k)	1,738	1,376
Shear Capacity (k)	4,719	4,719
Shear Demand/Capacity	0.37	0.30
Design Moment Demand per pile (k-in)	191,146	151,320
Moment Capacity per pile	309,907	245,915
Moment Demand/Capacity	0.617	0.615

2.4 Limits of Standard Bridge Design and Special Bridge Design

The boundary spans as mentioned in Section **Error! Reference source not found.** have the tandard span length and cross section, and are considered as standard structures. Therefore, the standard bridge design is suitable for use on spans 1 to 13 and from spans 15 to 32. The crossover structure itself occupies the section of viaduct between bent 14 and bent 15 and is the subject of this analysis.

2.5 Construction Methods Assessment

The assumed method and sequence of construction for the crossover structure is to construct the CIDH shafts alongside the BNSF right-of-way line. These piles will be extended as columns in a second stage concrete pour. Subsequently it is assumed that the lower part of the column cap beam will be formed and cast on falsework to provide a temporary seat onto which the precast beams can be placed.

Each beam will have a lift weight of approximately 60 to 70 tons and the erection lift radius is likely to be approximately 100 to 130 feet.

It is assumed that beams will be lifted from the east side of the structure straight from the delivery truck using a mobile crane. It is expected that as beam placement is a relatively quick operation, this can be done between trains running on the BNSF, though the BNSF should be consulted to confirm the acceptability of this approach. Some beams adjacent to expansion joints may require additional concrete for the joints to be cast onto them as a second stage pour prior to erection.

Once a section of beams between expansion joints is placed, the deck slab in that area can be cast to produce the final slab structure. Stay in place forms soffit forms will be required between beams per BNSF guidelines. The deck pour is also assumed to include the upper half of the column capping beam which allows the beams and deck to act monolithically with the column cap and columns.

The constraints specific to the crossover structure suggest that a particular method of erection is most likely to be used by contractors. This does not rule out other methods of construction. It is likely that contractors will prefer to use methods that they have used successfully in the past. The assessment described here represents a subset of methods that could be used.

2.6 Temporary Construction Loadings Considered

No specific loadings have been considered for the temporary stages described.

2.7 Temporary Construction Easements

A general temporary construction easement of 100 feet width has been identified on one side of the crossover structure with a 10-foot width on the other side. These TCEs extend for the full length of the crossover structure. The side of the structure that has the 100-foot width was chosen as the side that appears to have easiest connection to the local roadway network. It is expected that a 100-foot TCE will be sufficient to accommodate the access and crane requirements for beam placement.

Provision has been made for temporary construction easements of 15 feet width to both sides of the proposed HSR right-of-way boundary where the standard viaduct is used.

2.8 Traffic or Pedestrian Diversion and Control

The construction of the standard viaduct is expected to be supplied from along the route. At the crossover structure there is access to the local roadway system via E Conejo Ave and S Topeka Ave and it is expected that these routes will be the primary means for supplying beams and construction materials to the worksite. As this is a rural area it is anticipated that only localized traffic control measures will be required at the site entrances at peak work times.

2.9 Drainage Concept

The track drainage for the Conejo Crossover Structure will be carried from deck level through to a permanent drainpipe fitted within the void of the concrete deck girders. This pipe will be connected to downpipes cast into the columns. The downpipes will outfall near ground level to the surface drainage system.

For the crossover structure, provision will be made for collecting water at track level. This will be conveyed to the ends of each thermal unit of the deck slab via a longitudinal carrier pipe that will be located within the track bed. At the ends of the thermal unit carrier pipes will direct flow towards the edge beams and discharge through the expansion joints to the nearest available downpipe.

2.10 Emergency Access Provision

Provision for emergency access will be made in accordance with TM 2.8.1 Safety and Security Design Requirements R0 (March 12, 2012), TM 2.3.3 HST Aerial Structure R0 (June 2, 2009), NFPA130 and NFPA101. Emergency access points are required at maximum 2,500-foot intervals along aerial structures with access stairs to be located every 2.5 miles. It is also a requirement that access to the trackside is provided at each systems site which are also at approximately 2.5 mile intervals. Therefore, access stairs have been provided at each systems site and emergency ladder access turnarounds are provided at 2,500-foot nominal centers between systems sites. Due to the length of the viaduct and the spacing of access stairs on the route, stairs are not required on the Conejo Crossover Structure.

Table 2.10-1
Access Locations

STA	Locale	Egress features
1111+00	Adjacent to BNSF, north of E Conejo Ave	Ladder Access Turnaround
1146+30	Adjacent to S Peach Ave	Ladder Access Turnaround

2.11 Inspection, Service, and Maintenance Access

The standard viaduct will be a simple concrete section which can be inspected from both inside and outside. Inspection hatches will be located near the ends of girders.

The crossover beams are envisioned to be placed immediately adjacent to one another and cast into the edge beam at their ends. These are unlikely to be inspectable as they will also be over the BNSF corridor and they should be designed with this in mind. There will need to be an agreement between the Authority and the BNSF to provide access for future inspection and maintenance during non-revenue hours.

Externally, the crossover structure will be inspectable with the use of hydraulic access platforms either from grade or above.

2.12 Utilities Affected and Disposition

Refer to composite utility plans.

2.13 Noise Mitigation and Acoustic Treatment

No specific features have been included to mitigate the noise generated by the passage of trains. However, this is a rural area and it is considered unlikely that there will be many noise sensitive receptors sufficiently close to the structure to benefit from mitigation measures. The viaduct parapets are capable of carrying noise barriers if mitigation is required.

2.14 Compliance with System-Wide Bridge Aesthetics Features

TM 200.06, "Aesthetic Guidelines for Non-Station Structures" provides guidance on the appearance targets for the CHSTP. The scheme detailed on the PE4P drawings and analyzed represents the functional baseline case on which the DB contractors are encouraged to improve in discussion with the Authority.

2.15 Geotechnical Parameters Used for Design

The geotechnical parameters are described in the Geotechnical Design Report attached at Appendix A.

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Section 3.0

Dutch John Cut

3.0 Dutch John Cut Bridge

Dutch John Cut is one of three channels to the east of Hanford which carry the Kings River flow. The HSR route crosses Dutch John Cut on the Kings River Viaduct and at the point of crossing this is a two-span steel truss.

3.1 Structure Form

Constraints imposed by the US Army Corps of Engineers (USACE) on acceptable clearance from the river levees at this location have dictated the use of two 357-foot spans as indicated in the Figure 3.1-1 below.

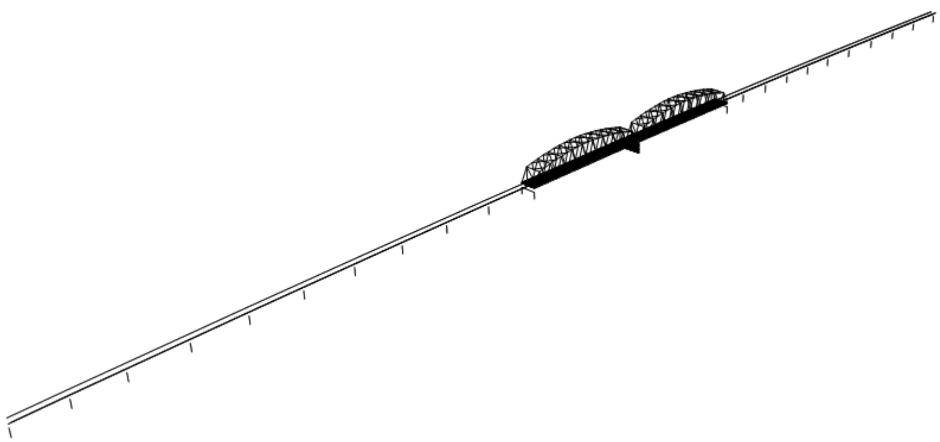

Figure 3.1-1
SAP Model

The overall length between bent centerlines of this section of the viaduct is assumed to be 714 feet making some allowance for additional joint gaps at supports.

The truss follows the same overall style as used for CP 1C. The top chord member for each span is a steel box section that increases in overall depth at each node connection to a maximum of 57 feet (centerline of top chord to centerline of bottom chord). The structure depth is similar to the shape of the bending moment diagram so that chord stresses are relatively uniform throughout the span. A reinforced concrete deck slab spans between the floor beams. This acts compositely with the bottom chord and also the transverse floor beams.

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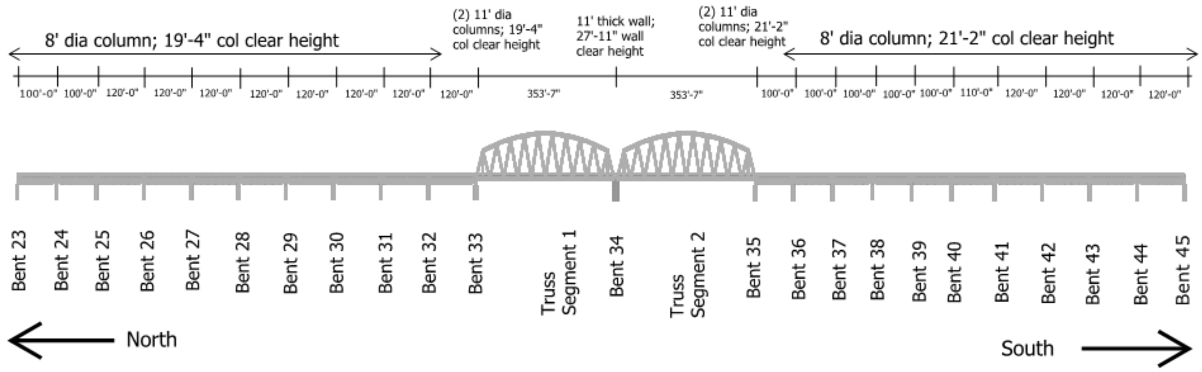

Figure 3.1-2
Column and Span Geometry

The substructure pier bents for bents 33 and 35 have been modeled as two 11-foot column pier bents. At bent 34 a two rectangular column bent, each column measuring 5 by 18 feet has been assumed to provide adequate stiffness. Bents 33 and 35 are supported by a pile cap with 6 No 6-foot-6-inch-diameter CIDH piles, and bent 34 is supported by a pile cap with 7 No 6-foot-6-inch-diameter CIDH piles.

3.2 Structure Importance Classification

TM 2.3.2 paragraph 2.2.1 defines all structures supporting the high-speed tracks to be primary structures because they will be required to be reinstated to allow resumption of train service after an earthquake.

This classification implies the following:

- Design life is 100 years.
- Seismic design must comply with TM 2.10.4.
- When applying the AASHTO LRFD code, values for the importance, ductility, and redundancy factors, hI, hD, and hR have been chosen as follows:
  - Importance factor hI = 1.05.
  - Ductility factor hD = 1.05 for strength calculations.
  - Redundancy factor h_R = 1.05 for nonredundant elements, 1.0 otherwise.

3.3 Key Design Features and Site Constraints

3.3.1 USACE Levees

The US Army Corps of Engineers (USACE) has responsibility for the maintenance of the levees to either side of the channel of Dutch John Cut. Their requirements are that no part of any structure crossing the levee is permitted within 15 feet of the outer toe of the levee. This has been discussed in the *15% Record Set Advance Planning Study Report* and has resulted in the decision to use two 350-foot spans for this crossing.

3.3.2 Ground Conditions

Geotechnical advice is based on historic borehole records from Caltrans projects located in the vicinity of the route, as no local borehole data were available. The foundation spring stiffness has therefore been based upon the lower-bound interpretation of the soil parameters, using the nearest borehole data and engineering judgment. Detailed design will be based on investigation results which are expected to demonstrate that this approach is conservative.

See the Geotechnical Design Report in Appendix A for details.

3.4 Summary of Analysis and Results

The PC box girder spans before and after the two truss spans are standard structures and do not fall within the scope of the preliminary design, although they have been modeled where necessary in accordance with the seismic design criteria, to ensure that the structure behavior of the truss spans is fully representative.

All sections have been checked for resonance effects, rail serviceability and track-structure interaction limits, and force demands. In all cases the structure has been found to be satisfactory.

Based upon the calculations the preliminary designs are in full compliance with the TMs with the exception of the 330-foot maximum span requirement and are capable of being developed into an acceptable design solution.

A design variance request has been submitted in support of the use of a 350-foot span.

The main results are summarized in Table 3.4-1 to 3.4-12.

3.4.1 Modeling

Both SAP and CSiBridge modeling programs were used for the analysis of Dutch John Cut. Several models of each section were required in order to represent the different conditions of the structure at different loading cases and for different design checks, in accordance with TM 2.10.4 and 2.10.10.

The structural columns, cross beams, rails and RC girders were represented by stick elements. Piles were represented by nonlinear springs, using equivalent stiffness values to correctly model the soil structure interaction based upon soil parameters in the GDR (Appendix A). The pile cap and pile group effects were modeled using rigid links connecting the top of the piles to the pier bent elements. All viaduct spans were connected to the bent cap elements with linear bearing springs, with the bridge articulation represented by either pin or roller spring properties. In the case of the transverse frequency analysis, pinned restraints were added in place of the bearings, as only the flexibility of the superstructure is to be considered.

Foundation arrangements for the standard spans were those used in the Authority's representative's design for the standard viaducts and have been used accordingly in the structural models. These foundations have been checked using LPILE and Pilset and found to have adequate capacity.

3.4.2 Frequency Results

The vertical, torsional, and transverse frequencies of the structure were evaluated to ensure that they meet the required dynamic criteria, as defined by TM 2.10.10. In the case of the vertical and torsional frequencies, two conditions were assessed: the first with upper bound mass and lower

bound stiffness (Condition 1); the second with lower bound mass and upper bound stiffness (Condition 2). Condition 1 was also adopted for the transverse frequency analysis, as this generated the most onerous frequencies in this case.

The natural frequencies were found to be within the defined limits. The effective length used for frequency analysis is the truss span of 280 feet. For limits and natural frequencies of each thermal span, see the structural calculations. See Table 3.4-1 for a summary of the results.

Table 3.4-1
Dutch John Cut Bridge Frequency Check Results

	Vertical Frequency (Hz)	Torsional Frequency (Hz)	Transverse Frequency (Hz)
Lower Limit	1.47	1.91 condition 1 2.27 condition 2	1.2
Upper Limit	2.88	N/A	N/A
Condition 1	1.59	2.37	2.12
Condition 2	1.89	2.71	N/A

3.4.3 Rail Serviceability and Track-Structure Interaction Results

The truss spans were analyzed for deflections and rail stresses and evaluated against the limits prescribed in TM 2.10.10. This included the assessment of global deflections of the structure, the relative rotations and displacements at the rails and expansion joints, and the relative twist of the deck.

Several SAP models were developed to model train loads at the most onerous locations on or immediately adjacent to, the truss spans. The various train locations were coupled with the load permutations and cases specified in TM 2.10.10 to envelope the worst deflections in the structure.

It has been demonstrated from the analysis that the truss spans meet all of the requirements of TM 2.10.10 with the exception of maximum span, and that all rotations, deflections and rail stresses are within limits. See Tables 3.4-2 to 3.4-8 for a summary of the results. Results shown are for the worst cases only. Joint-specific results can be found in the complete calculation report along with supporting calculations.

Table 3.4-2
Dutch John Cut Bridge Track Serviceability Results (1)

Group	Vertical Deflection L = 350 ft		Transverse Deflection L = 350 ft	
	Limit	**Dutch John Cut Bridge**	**Limit**	**Dutch John Cut Bridge**
Group 1a	2.64	0.76	1.724	0.070
Group 1b	3.85	1.54	3.334	0.078
Group 3	N/A	N/A	5.387	0.643

Table 3.4-3
Dutch John Cut Bridge Track Serviceability Results (2)

Group	Rotation about Vertical Axis (rad)		Rotation about Transverse Axis (rad)	
	Limit	Dutch John Cut Bridge	Limit	Dutch John Cut Bridge
Group 1a	0.0007	0.00017	0.0012	0.00091
Group 1b	0.0010	0.00008	0.0017	0.00168
Group 3	0.0021	0.00061	0.0026	0.0017

Table 3.4-4
Dutch John Cut Bridge Track Serviceability Results (3)

Group	Deck Twist (Rad)	
	Limit	Dutch John Cut Bridge
Group 1a	0.0012	0.00026
Group 1b	0.0012	0.00046
Group 3	0.0034	0.00045

Table 3.4-5
Dutch John Cut Bridge Track Structure Interaction Results (1)

Group	Relative Longitudinal Displacement at Expansion Joints (in)	
	Limit	Dutch John Cut Bridge
Group 4	2.1	1.302
Group 5	2.880	0.883

Table 3.4-6
Dutch John Cut Bridge Track Structure Interaction Results (2)

Group	Relative Vertical Displacement (in)	
	Limit	Dutch John Cut Bridge
Group 4	0.25	0.075
Group 5	0.50	0.201

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Table 3.4-7
Dutch John Cut Bridge Track Structure Interaction Results (3)

Group	Relative Transverse Displacement (in)	
	Limit	Dutch John Cut Bridge
Group 4	0.08	0.041
Group 5	0.16	0.025

Table 3.4-8
Dutch John Cut Bridge Track Structure Interaction Results (4)

Group	Permissible Axial Rail Stress (ksi)	
	Limit	Dutch John Cut Bridge
Group 4	±14	9.82 to -9.42
Group 5	±23	20.33 to -22.68

3.4.4 Force Results

The key components of the Dutch John Cut Bridge have been checked for structural adequacy to assess the validity of the section sizing. Sections of the standard viaduct which interface with the truss were also checked.

The force checks included the RC section design of the columns and pile caps, stress checks of the truss members and redundancy checks of the truss to assess the no collapse condition on loss of a critical member.

Table 3.4-9
Column Strength Check – Load Case, Axial, and Flexural at governing locations

	11-Foot Column (SRSS)		5-Foot by 18-Foot Column (weak dir)		8-Foot Column (SRSS)	
Strength Load Combination	**1**	**5**	**1**	**5**	**1**	**5**
Axial Demand (k)	7,675	4,747	13,185	18,135	5,857	4,703
Moment Demand (k-in)	296,949	382,305	8,562,300	713,525	198,147	283,431
Shear Demand SRSS (k)	1,077	504	1,380	2,160	312	815
Moment Demand/Capacity Ratio	0.461	0.600	0.542	0.336	0.554	0.936

Table 3.4-10
Pile Strength Check – Governing Axial/Moment Load per Pile

	Bent 33,35	♦ **Bent 34**
Governing Axial Demand(k)	1,350	2,370
Shear Demand SRSS (k)	399	333
Flexural Demand(k-in)	28,500	19,128
Moment Demand/Capacity Ratio	0.221	0.131

3.4.5 Seismic Results

3.4.5.1 Columns

The displacement capacity and displacement ductility demand of the columns were assessed in accordance with the CSDC. 1.2% reinforcement ratio is provided for the 11-foot truss column. The reinforcing steel detail is shown in Figures 3.4-1. The assessment was completed using a combination of global and local SAP models with pushover analyses in both the X and Y directions.

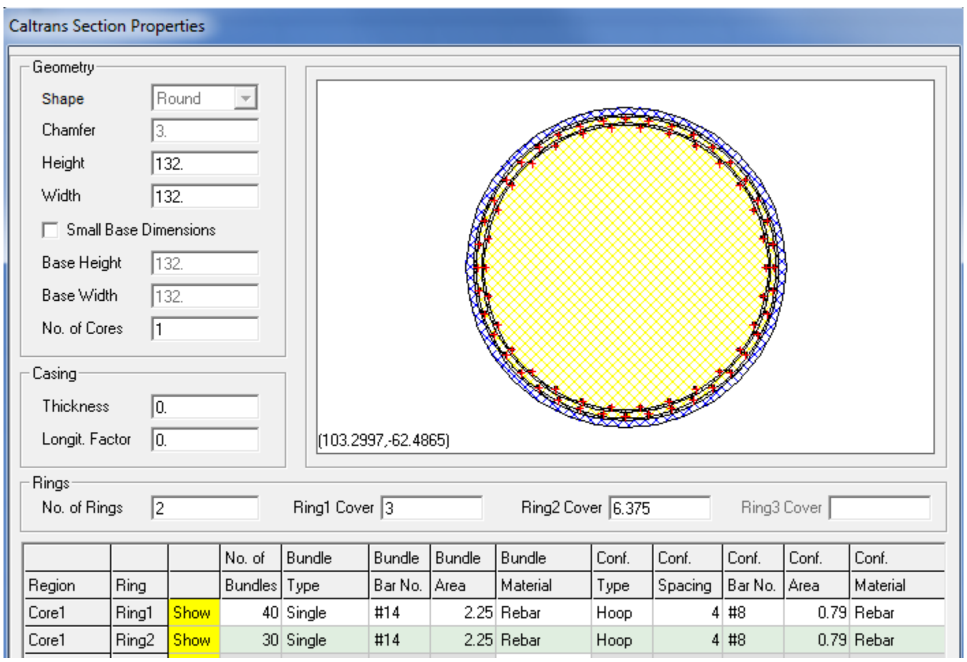

Figure 3.4-1
Section Properties – Dutch John Cut Bridge 11-Foot Truss Column

Bent 34 is analyzed with 2-foot-5-inch by 18-foot rectangular columns framed in transverse direction. The longitudinal stiffness of this bent configuration matches original 11 by 45-foot pier wall in SAP model with 37.5-foot height columns. See Figure 3.4-2 for section properties.

Section Details:

X Centroid:	.4532E-14 in
Y Centroid:	-13.89E-3 in
Section Area:	12.96E+3 in^2
EI gross about X:	1.91E+11 kip-in^2
EI gross about Y:	1.50E+10 kip-in^2
I trans (Confined1) about X:	5.30E+7 in^4
I trans (Confined1) about Y:	4.158E+6 in^4
Reinforcing Bar Area:	71.85 in^2
Percent Longitudinal Steel:	.5544 %
Overall Width:	60.00 in
Overall Height:	216.0 in
Number of Fibers:	810
Number of Bars:	46
Number of Materials:	3

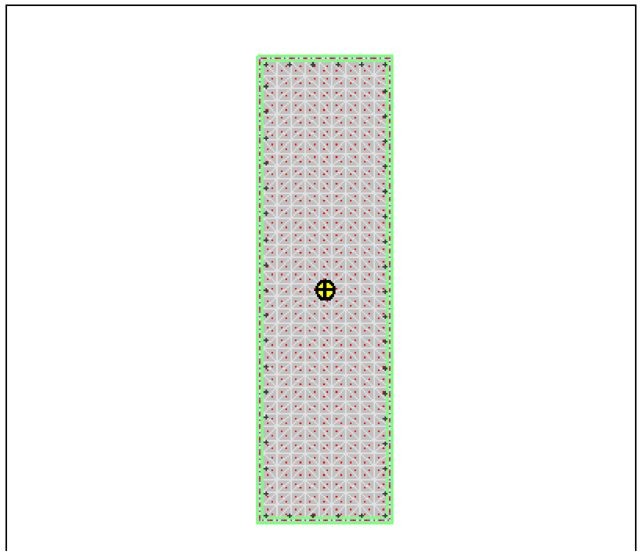

Figure 3.4-2
Section Properties - Dutch John Cut Bridge 5 by 18-Foot Leaf Pier Column

It has been demonstrated from the analysis and calculations that Dutch John Cut Bridge is structurally viable and meets the requirements of the CSDC. For bent-specific demands, see the structural calculations. See Table 3.4-11 to Table 3.4-14 for a summary of the results.

Table 3.4-11
Seismic Displacement Check – Displacement Demand Ductility Check

	Displacement Ductility Upper Limit	MCE Displacement ΔD (in)	Yield Displacement ΔY (in)	Displacement Ductility, μD
11-foot Column Longitudinal	5	0.75	0.60	1.25
11-foot Column Transverse	5	0.72	2.50	0.29
5-foot by 18-foot Column Longitudinal	5	1.52	1.86	0.81
5-foot by 18-foot Column Transverse	5	1.40	2.46	0.57

Table 3.4-12
Seismic Displacement Check – Capacity Ductility Check

	Capacity Ductility Lower Limit	Yield Column Displacement ΔYCOL (in)	Collapse Column Displacement ΔcCOL (in)	Capacity Ductility, μC
11-foot Column Longitudinal	3	0.17	1.78	10.47
11-foot Column Transverse	3	0.178	1.7	9.55
5-foot by 18-foot Column Longitudinal	3	0.75	4.45	5.09
5-foot by 18-foot Column Transverse	3	1.36	11.37	8.34

Table 3.4-13
Seismic Displacement Check – Displacement Demand/Capacity Ratio Check

	MCE Displacement Demand ΔD (in)	Displacement Capacity Δc (in)	Demand/Capacity
11-foot Column Longitudinal	0.750	2.38	0.32

	MCE Displacement Demand ∆D (in)	Displacement Capacity ∆c (in)	Demand/Capacity
11-foot Column Transverse	1.400	5.81	0.24
5-foot by 18-foot Column Longitudinal	1.52	5.68	0.27
5-foot by 18-foot Column Transverse	1.40	12.54	0.11

Table 3.4-14
Column Strength Check

	11-foot Column	8-Foot Column	5-foot by 18-foot Column (longitudinal)	5-foot by 18-foot Column (transverse)
Plastic Moment, Mp (k-in)_	693,467	357,602	1,062,000	353,400
Overstrength Moment, Mo (k-in)	832,160	429,122	1,274,400	424,080
Overstrength Shear Demand, V (kips)	6,604	1,703	2,832	2,019
Shear Capacity ØVn (kips)	7,664	4,054	5,410	5,410

3.4.5.2 Foundations

The demand forces to be used for the foundation design are specified in the CSDC and include the service level moments, shears, axial loads and the moment demand induced by the column plastic hinging mechanism. The moment derived from the plastic hinging mechanism is taken as the plastic moment of the column, multiplied by an overstrength factor of 1.2 to give the overstrength moment.

The adoption of the overstrength factor is based upon the seismic design philosophy whereby the columns will always yield to protect the pile. The factor is required to account for risk that the column may develop a greater plastic moment capacity than the idealized values used in the design. An axial-moment diagram from the section designer module in CSiBridge was used to check the reinforcement design of the pile. A reinforcement ratio of 2% and #8 at 12-inch tie spacing was used for 6.5ft diameter pile. Pile shear and moment demand derived from column overstrength moment are checked against capacity.

See Table 3.4-15 for a summary of the results. For bent locations, see Figure 3.1-1.

Table 3.4-15
Pile Strength Check – Column Overstrength Demand per Pile

	Bents 33, 35	Bent 34 Transverse	Bent 34 Longitudinal
Extreme 3 Axial Demand (k)	-597 (tension)	4,524	4,088
Overstrength Moment Demand (k-in)	33,108	73,138	102,567
Moment Capacity (k-in)	175,000	247,000	243,000
Moment D/C	0.19	0.30	0.42
Overstrength Design Shear Demand (k)	1518	812	1140
Shear Capacity (k)	2676	2676	2676
Shear D/C	0.57	0.30	0.43

The pile cap must transfer forces from the column into the piles. The column's overstrength moment was used to find the demand on the pile cap. A moderate amount of flexural reinforcing steel in the pile cap, three rows of #11 bars at 6 inches each way at the bottom and #11 bars at 6 inches each way at the top, was sufficient for demands. Vertical shear reinforcement of #8 bars at 12 inches for the column pile cap was needed for shear in the pile cap. See Table 3.4-16 for a summary of the results for the governing load cases of the column pile cap. For bent locations, see Figure 3.1-1.

Table 3.4-16
Pile Cap Strength Check

	Bent 33,35	Bent 34
Max Moment Demand at Column Face per Foot (k-ft)	2,624	3207
Moment Capacity at Column Face per Foot (k-ft)	3,279	3,268
Max. Shear Demand, "d" Away from Column per Foot (k)	389	226
Shear Capacity, "d" Away from Column per Foot (k)	480	481
Max. Punching Shear Demand at Pile (k)	6,246	4,905
Punching Shear Capacity at Pile (k)	11,683	11,711

3.5 Limits of Standard Bridge Design and Special Bridge Design

Dutch John Cut Bridge is located at spans 45 and 46 of the Kings River Viaduct and meets the criteria for a Complex and Non-standard structure in TM 2.10.10. Similar truss spans are located at spans 2, 19, 95, 96 and 102 of the viaduct. It is assumed that the standard viaduct design is suitable for use on spans 1, 3 to 18, 20 to 44, 47 to 94 and 97 to 101.

3.6 Construction Methods Assessment

Given its location in the middle of a long viaduct of standard precast spans and the configuration of the river channel at the crossing site, it is expected that either of the following will occur:

- The two truss spans would be constructed in their final position by assembling the major members on site from large prefabricated sub-elements.
- The superstructure of the span that crosses the flowing river channel could be constructed at the adjacent span prior to launching across the river channel using temporary supports. The second span would then be constructed in the same way as the first but would be in its final position.

Neither of these erection methods rules out other methods of construction from consideration.

It is likely that contractors will prefer to use methods that they have used successfully in the past. The assessment described here therefore represents a subset of methods that could be used.

3.7 Temporary Construction Loadings Considered

No specific loadings have been considered for the temporary stages described.

3.8 Temporary Construction Easements

A general temporary construction easement of 15 feet width outside of the permanent right-of-way has been indicated for the full length of the viaduct. At the structure location this gives a total corridor width of 100-feet which is considered sufficient for the foreseeable requirements for construction of such a structure. However, should more space be required, the contractor has the option to negotiate additional easements with adjacent landowners.

3.9 Traffic or Pedestrian Diversion and Control

The location of these spans in in a very rural and lightly trafficked location, so it is not expected that elaborate measures for control of traffic or pedestrians will be required, though maintenance access for the maintenance of the USACE levees should be provided at all times through construction.

3.10 Drainage Concept

The track drainage for the Kings River Viaduct will be carried from deck level through to a permanent drainpipe fitted within the void of the concrete deck girders. This pipe will be connected to downpipes cast into the columns. The downpipes will outfall near ground level to the surface drainage system.

For the Dutch John Cut Bridge spans, provision will be made for collecting water at track level. This will be conveyed to the ends of each span via a longitudinal carrier pipe located within the deck floor beams. At the ends of the span water will be discharged through the abutment or piers to the surface drainage system. It is not intended that the viaduct will discharge directly to the river.

3.11 Emergency Access Provision

Provision for emergency access will be made in accordance with TM 2.8.1 and NFPA130. Emergency access points are required at maximum 2,500-foot intervals along aerial structures with access stairs to be located every 5 miles. It is also a requirement that access to the trackside is provided at each systems site which are also at approximately 5 mile intervals. Therefore, access stairs have been provided at each systems site and emergency ladder access turnarounds are provided at 2,500-foot nominal centers between systems sites. The nearest access turnarounds to Dutch John Cut Bridge are tabulated below in Table 3.11-1.

Table 3.11-1
Access Locations

STA	Locale	Egress features
1510+00	Near access road to SR 43	Ladder Access Turnaround
1535+00	Near to Ninth Ave	Ladder Access Turnaround

3.12 Inspection, Service, and Maintenance Access

The standard viaduct will be a simple concrete section that is inspectable from both inside and outside.

Dutch John Cut Bridge will have steel truss girder spans which are largely composed of hollow box sections. It is expected that these box sections will be accessible through access covers at key locations for inspection and maintenance purposes. If the specification permits, it is also possible that the DB Contractor may propose that the box sections are fully sealed to prevent moisture ingress, in which case the internal surfaces will not be inspectable without additional work to break and repair the seals. The external surfaces of the truss members will are accessible for inspection using a hydraulic access platform subject to suitable safety procedures for working at the trackside and also for working over and in the Dutch John Cut river channel.

3.13 Utilities Affected and Disposition

It is not anticipated that there are any major utilities in this area.

3.14 Noise Mitigation and Acoustic Treatment

No specific features have been included in the structure to mitigate the noise generated by the passage of trains. The project environmental impact statement (EIS) will define what measures are required to mitigate noise impacts.

However, this structure is considered sufficiently robust to accommodate the addition of, for example, noise protection fencing, should this be required for mitigation of impact.

3.15 Compliance with System-Wide Bridge Aesthetics Features

TM 200.06, “Aesthetic Guidelines for Non-Station Structures” provides guidance on the appearance targets for the CHSTP. The scheme detailed on the PE4P drawings and analyzed represents the functional baseline case on which the DB contractors are encouraged to improve in discussion with the Authority.

3.16 Geotechnical Parameters Used for Design

The geotechnical parameters are described in the Geotechnical Design Report attached at Appendix A.

Section 4.0

Kings/Tulare Regional Station

4.0 Kings/Tulare Regional Station

The Kings/Tulare Regional Station structure is conceived as a series of wide in situ concrete multicellular box girders which are either simply supported or continuous over a limited number of spans (typically a 3 span frame). These girders have been detailed at 10.5 feet deep to match the standard 30% viaduct section. Typically the spans are 120 feet, but also vary up to 140 feet to accommodate SR 198 and the Cross Valley Railroad, which cross the route at the south end of the station area. The longer spans are able to carry the HSR route at the same depth because the superstructure is relatively lightly loaded compared to the standard viaduct.

4.1 Structure Form

In plan the station area widens out abruptly to a width that will accommodate the additional platform tracks which serve the station, but also allow space for a storage track at each end of the platform area. At the north end of the station, the storage track is located on the east of the through tracks and at the south end of the station the storage track is located to the west. This arrangement makes the station structure rotationally symmetric in plan about the center of the station. The presence of the storage track makes the structure cross-section asymmetrical to the centerline of the route on the approaches to the platform area, but the section in the platform area itself is symmetrical.

Typical views and cross-sections of the structure model are shown in Figures 4.1-1 to 3 below.

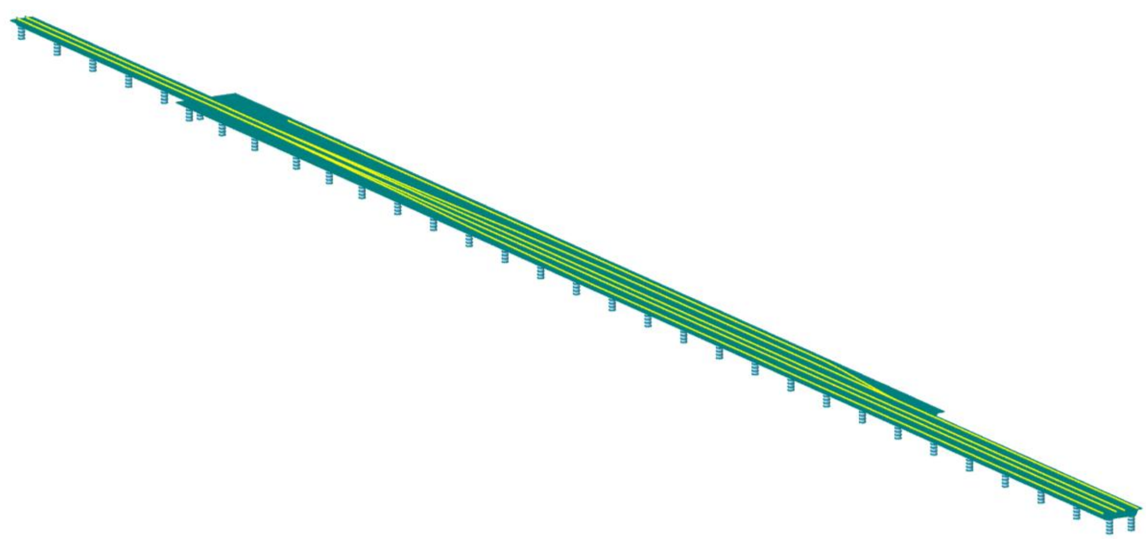

Figure 4.1-1
Midas Model – Isometric view

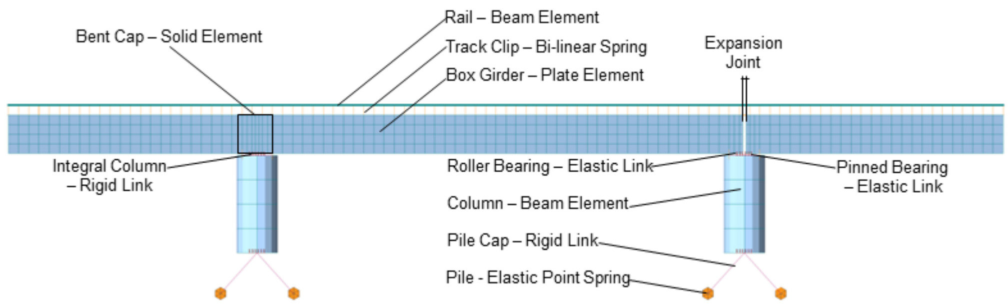

Figure 4.1-2
Midas Model – Single span detail

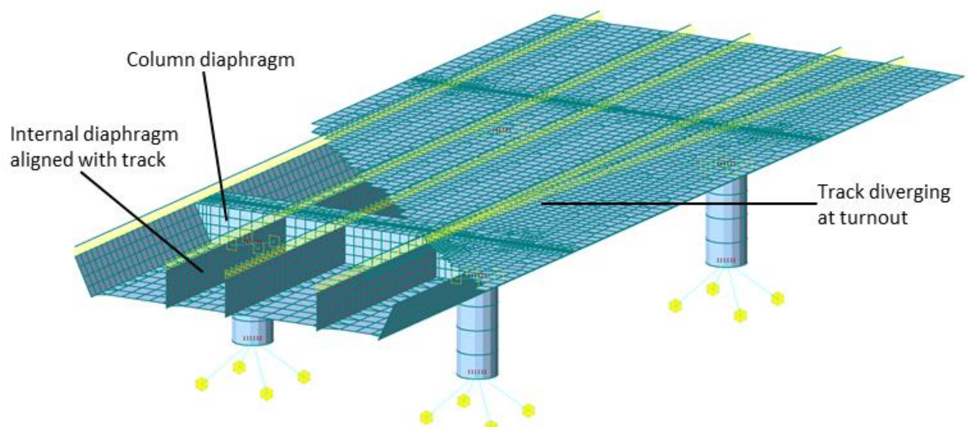

Figure 4.1-3
Midas Model – Deck and track details

4.2 Structure Importance Classification

TM 2.3.2 paragraph 2.2.1 defines all structures supporting the high-speed tracks to be primary structures because they will be required to be reinstated to allow resumption of train service after an earthquake.

This classification implies the following:

- Design life is 100 years.
- Seismic design must comply with TM 2.10.4.
- When applying the AASHTO LRFD code, values for the importance, ductility, and redundancy factors, hI, hD, and hR have been chosen as follows:
  - Importance factor hI = 1.05.
  - Ductility factor hD = 1.05 for strength calculations.
  - Redundancy factor h_R = 1.05 for nonredundant elements, 1.0 otherwise.

4.3 Key Design Features and Site Constraints

4.3.1 Ground Conditions

The geotechnical parameters used for the analysis are based on historic borehole records from Caltrans projects located in the vicinity of the route, as no project-specific or local borehole data were available. The foundation spring stiffness has therefore been based upon the lower-bound interpretation of the soil parameters, using the nearest borehole data and engineering judgment. Detailed design will be based on investigation results which are expected to demonstrate that this approach is conservative.

See the GDR in Appendix A for details of parameters and spring stiffness used in the analysis.

4.4 Summary of Analysis and Results

The Kings/Tulare Regional Station structure was modeled using shell elements to model the cellular superstructure with solid elements to represent transverse diaphragms at bent locations and beam elements to represent the columns. The span arrangement is typically 120-foot simply supported spans with some spans of 140 feet to cross obstacles such as SR 198 and the Cross Valley Railroad. In order to achieve these spans, the superstructure has been made continuous to make three- and five-span frames. The superstructure has been made continuous in other sections to avoid having a structure joint within the "no joint" zone around the proposed switches and turnouts for the platform and storage tracks.

PC box girder spans before and after the station slab area are standard structures and do not fall within the scope of the preliminary design. The global structure column arrangement is rotationally symmetric. Therefore, only half of the station is modeled. The southern half, which includes the Cross Valley Railroad and SR198, is likely to produce more severe results. Five spans of standard viaduct have been included in the model in accordance with the seismic design criteria, to ensure that the structure behavior of the station spans is fully representative and end effects are remote from the areas of interest.

All sections have been checked for fundamental frequency limits, rail serviceability and track-structure interaction limits, and column force demands. In all cases the structure has been found to be satisfactory.

Based upon these calculations the preliminary design is in full compliance with the TMs and is capable of being developed into an acceptable design solution. Refer to the Package 2-3 Structures Calculations report for the complete analysis and results.

The main results are summarized in Tables 4.4-1 to 4.4-14.

4.4.1 Modeling

MIDAS Civil 2013 version 3.1 was used for the analysis of Kings/Tulare Regional station structure. Due to the scale of the structure only one half of the station and leading standard viaduct spans were modeled for analysis. The station structure is rotationally symmetric in plan about the center of the station. It was considered that the southern half, which includes the longest spans and thermal units over the cross valley railroad and SR198, was likely to produce more onerous results. Five spans on either end of the model were considered neglected as boundary spans and rail stress was confirmed to dissipate before reaching the model bounds. Model ends are simply supported spans; therefore, no boundary stiffness springs were applied to the superstructure at the model boundary. The platform area is in an area where the structure is uniform and so of secondary interest in identifying the limiting conditions for rail stress, displacements etc.

Several models of each section were required in order to represent the different conditions of the structure for each load condition and for different design checks. All loadings have been applied in accordance with TM 2.10.4 and 2.10.10 as outlined in the Seismic Analysis Design Plan (see Appendix B).

The superstructure is a multicellular box girder and has been modeled using shell elements. The diaphragm beams at pier bents have been modeled as solid elements and the structural columns as stick elements. Piles were represented by nonlinear springs, using equivalent stiffness values to correctly model the soil structure interaction based upon soil parameters in the GDR (Appendix A). The pile cap and pile group effects were modeled using rigid links connecting the top of the piles to the pier bent elements. All typical viaduct spans were connected to the bent cap elements with linear bearing springs, with the bridge articulation represented by either pin or roller spring properties. In the unique case of the transverse frequency analysis, pinned restraints were added in place of the bearings, as only the flexibility of the superstructure is to be considered. Note that for the type of structure under consideration, the fixity requirement of TM 2.10.10 fully restrains the superstructure from all transverse movement.

Foundation arrangements for the standard spans are those used in the Authority's representative's design for the standard viaducts and have been used accordingly in the structural models. These foundations have been checked using LPILE and Pilset and found to have adequate capacity.

Analysis results for Bent 60 to Bent 65 are reported in Section 4.4.2 to Section 4.4.5.

Analysis results for Bent 44 to Bent 47 are reported in Section 4.4.6 to Section 4.4.10.

4.4.2 Frequency Results

The vertical, torsional, and transverse frequencies of the structure were evaluated to ensure that they meet the required dynamic criteria, as defined by TM 2.10.10. In the case of the vertical and torsional frequencies, two conditions were assessed: the first with upper bound mass and lower bound stiffness (Condition 1); the second with lower bound mass and upper bound stiffness (Condition 2). Condition 1 was also adopted for the transverse frequency analysis, as this generated the most onerous frequencies in this case.

The natural frequencies were found to be within the defined limits. See Table 4.4-1 for a summary of the results for the most onerous 580 ft continuous span. The effective span length used for frequency analysis was 174 ft as per TM2.10.10 Section 6.8.2. For limits and natural frequencies of other thermal spans, see the complete structural calculations. No torsional frequencies were found below 5.246 Hz for condition 1 and 5.714 Hz for condition 2.

Table 4.4-1
Kings/Tulare Regional Station Frequency Check Results

	Vertical Frequency (Hz)	Torsional Frequency (Hz)	Transverse Frequency (Hz)
Lower Limit	2.247	5.246 condition 1 5.714 condition 2	1.2
Upper Limit	4.860	N/A	N/A
Condition 1	4.372	greater than 5.246	5.478
Condition 2	4.762	greater than 5.714	N/A

It has been found that the structure complies with the vertical frequency requirements of TM 2.10.10 with a reasonable margin. The section sizes specified are therefore larger than would be required and may be further optimized by the contractor.

As the vertical frequencies are governed in large part by the ground conditions, access to site-specific GI data may reveal more beneficial soil parameters, and permit further savings in the refinement of the design.

4.4.3 Rail Serviceability and Track-Structure Interaction Results

The model spans were analyzed for deflections and rail stresses and evaluated against the limits prescribed in TM 2.10.10. This included the assessment of global deflections of the structure, the relative rotations and displacements at the rails and expansion joints, and the relative twist of the deck.

Several MIDAS models were developed to model train loads at the most onerous locations on, or immediately adjacent to, the most critical joints in the structure. The various train locations were coupled with the load permutations and cases specified in TM 2.10.10 to envelope the worst deflections in the structure.

Group 1c applies to the Kings/Tulare Regional Station structure where the structure supports more than two tracks. For the continuous span of interest between bents 60 and 65, for which worst case results are presented in the following tables, three tracks are loaded in Group 1c: the two through tracks and one exterior storage track. Only vertical live load (no lateral live load) was applied to the storage track for Group 1c. When applying load combinations for OBE events, only half the number of total tracks shall be considered simultaneously (TM 2.10.4). Therefore, a maximum of two tracks were loaded for load Groups 3 and 5.

It has been demonstrated from the analysis that the spans meet most of the requirements of TM 2.10.10 except for the limit of rotation about transverse axis of the bridge. For case specific results at various bent locations, see the structural calculations report. See tables 4.4-2 to 4.4-8 for a summary of the results at the most onerous locations.

In order to satisfy the rotation limit, dowel connections are used at the expansion joint between the typical viaduct and storage track spans (Bent 65). The purpose of the dowels is to control the relative movement between the structural units and, in particular, the movement at the rails.

Table 4.4-2
Kings/Tulare Regional Station Track Serviceability Results (1)

Group	Vertical Deflection L = 140 ft		Transverse Deflection L = 140 ft	
	Limit	Kings/Tulare	Limit	Kings/Tulare
Group 1a	0.500	0.302	0.272	0.013
Group 1b	0.728	0.454	0.526	0.015
Group 1c	2.800	0.486	N/A	N/A
Group 3	N/A	N/A	0.850	0.180

Table 4.4-3
Kings/Tulare Regional Station Track Serviceability Results (2)

Group	Rotation about Vertical Axis		Rotation about Transverse Axis	
	Limit	Kings/Tulare	Limit	Kings/Tulare
Group 1a	0.0007	0.00012	0.0012	0.00086
Group 1b	0.0010	0.00003	0.0017	0.00178
Group 2	0.0021	0.00012	0.0026	0.00086
Group 3	0.0021	0.00186	0.0026	0.00100

Table 4.4-4
Kings/Tulare Regional Station Track Serviceability Results (3)

Group	Deck Twist	
	Limit	Kings/Tulare
Group 1a	0.06	0.037
Group 1b	0.06	0.051
Group 2	0.17	0.037
Group 3	0.17	0.105

Table 4.4-5
Kings/Tulare Regional Station Track Structure Interaction Results (1)

Group	Relative Longitudinal Displacement	
	Limit L = 290 ft	Kings/Tulare
Group 4	1.696	1.010
Group 5	3.026	0.655

Table 4.4-6
Kings/Tulare Regional Station Track Structure Interaction Results (2)

Group	Relative Vertical Displacement	
	Limit	Kings/Tulare
Group 4	0.25	0.083
Group 5	0.50	0.029

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Table 4.4-7
Kings/Tulare Regional Station Track Structure Interaction Results (3)

Group	Relative Transverse Displacement	
	Limit	Kings/Tulare
Group 4	0.08	0.022
Group 5	0.16	0.123

Table 4.4-8
Kings/Tulare Regional Station Track Structure Interaction Results (4)

Group	Permissible Axial Rail Stress	
	Limit	Kings/Tulare
Group 4	±14	5.7 to -6.7
Group 5	±23	7.4 to -3.0

4.4.4 Force Results

The key components of the station structure have been checked for structural adequacy to assess the validity of the section sizing. In addition to the station structure, columns supporting the spans of the typical viaduct were also checked.

The force checks comprised of the RC section design of the columns and pile caps and the RC design of the piled foundations.

Table 4.4-9
Column Strength Check – Load Case, Axial, and Flexural

	Strength 1	Strength 5
Axial Demand (k)	2,644	1,703
Shear SRSS (k)	786	1,110
Moment M3 Demand (k-in)	215,613	168,860
Moment M2 Demand (k-in)	3,714	134,749
Moment SRSS Demand/Capacity Ratio	0.60	0.55

Table 4.4-10
Pile Strength Check – Governing Axial/Moment Load per Pile

	Strength 1	Strength 5
Axial Demand (k)	304	174
Shear Demand SRSS (k)	87	51
Flexural Demand (k-in)	7791	4546
Demand/Capacity Ratio	0.14	0.08

4.4.5 Seismic Results

4.4.5.1 Columns

The displacement capacity and displacement ductility demand of the columns were assessed in accordance with the CSDC. One percent reinforcement ratio is provided for each column. The reinforcing steel detail is shown in Figure 4.4-1. The assessment was completed using a combination of global and local SAP models and running pushover analyses in both the X and Y directions.

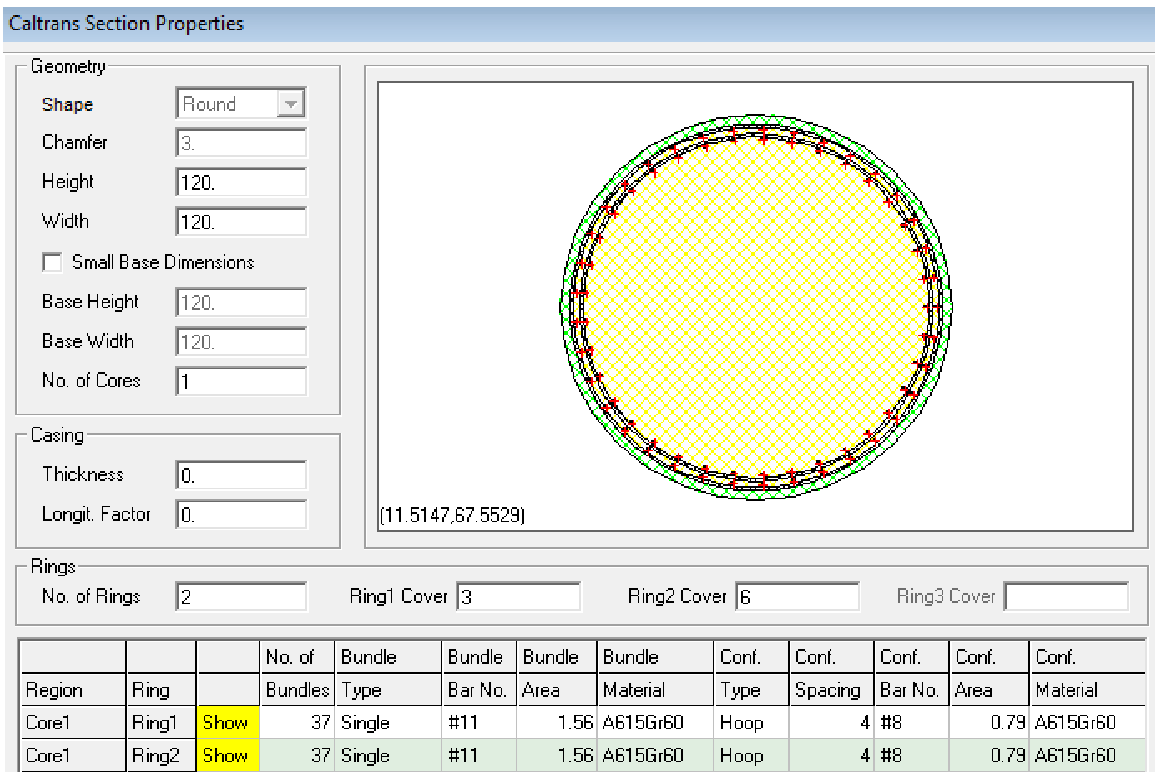

Figure 4.4-1
Kings/Tulare Regional Station 10ft Column Design

It has been demonstrated from the analysis and calculations that the Kings Tulare Station concept is structurally viable and meets the requirements of the CSDC. See tables 4.4-9 to 4.4-12 for a summary of the results.

Table 4.4-11
Seismic Displacement Check – Displacement Demand Ductility Check

	Displacement Ductility Upper Limit	MCE Displacement ΔD (in)	Yield Displacement ΔY (in)	Displacement Ductility, μD
Longitudinal	5	0.733	0.2659	2.8
Transverse	5	0.511	0.2956	1.7

Table 4.4-12
Seismic Displacement Check – Capacity Ductility Check

	Capacity Ductility Lower Limit	Yield Column Displacement ΔYCOL (in)	Collapse Column Displacement ΔcCOL (in)	Capacity Ductility, μC
Longitudinal	3	0.351	4.455	12.7
Transverse	3	0.319	4.423	13.9

Table 4.4-13
Seismic Displacement Check – Displacement Demand/Capacity Ratio

	MCE Displacement Demand ΔD (in)	Displacement Capacity Δc (in)	Demand/Capacity
Longitudinal	0.733	4.721	0.16
Transverse	0.511	4.719	0.11

Table 4.4-14
Column Strength Check

	Envelope
Plastic Moment, Mp (k-in)	536,880
Overstrength Moment, Mo (k-in)	644,256
Overstrength Shear Demand, V (kips)	1,255
Shear Capacity ØVn (kips)	6,910

4.4.5.2 Foundation Design

The demand forces to be used for the foundation design are specified in the CSDC and include the service level moments, shears, axial loads and the moment demand induced by the column plastic hinging mechanism. The moment derived from the plastic hinging mechanism is taken as the plastic moment of the column, multiplied by an overstrength factor of 1.2 to give the overstrength moment.

The adoption of the overstrength factor is based upon the seismic design philosophy whereby the columns will always yield at the MCE event to protect the pile. The factor is required to account for risk that the column may develop a greater plastic moment capacity than the idealized values used in the design.

This approach has also been used for the design of the piled foundations of the nonstandard columns immediately adjacent to the crossover structure. An axial-moment diagram from the section designer module in CSiBridge was used to check the reinforcement design of the pile. A minimal reinforcement ratio of 2% is used for the design for 6.5ft diameter pile. Pile shear and moment demand derived from column overstrength moment are checked against capacity.

Table 4.4-15
Pile Capacity Protection – Column Overstrength Demand/Capacity Ratio per pile

	Max Axial	Min Axial
Extreme 3 Axial Demand (k)	911	125
Overstrength Moment Demand (k-in)	127,049	99,272
Momend Capacity (k-in)	186,747	172,088
Moment D/C	0.54	0.46
Overstrength Design Shear Demand (k)	1,614	1,106
Sehar Capacity (k)	2,676	2,676
Shear D/C	0.53	0.41

4.4.6 Modeling at Cross Valley Railroad Spans

A 230-ft-span structure at Cross Valley Railroad is part of the analysis model addressed in Section 4.4.1. Analysis results from Bent 44 to Bent 47 are reported in Section 4.4.7 to Section 4.4.10.

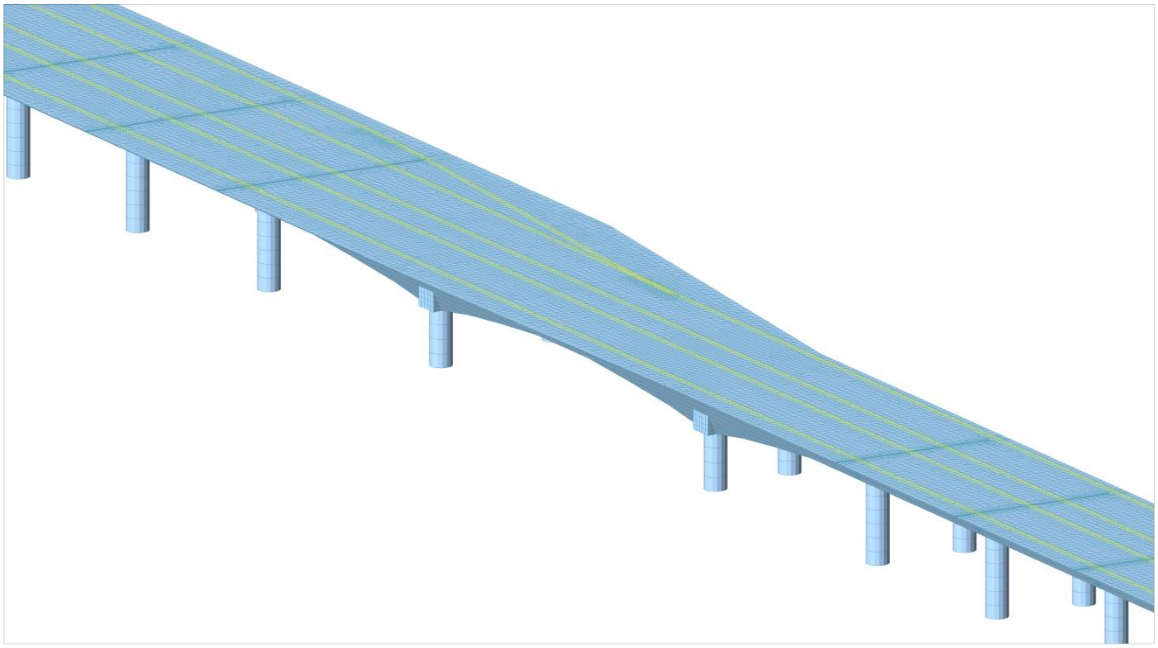

Figure 4.4-2
Isometric View of Model

4.4.7 Frequency Results

The vertical, torsional, and transverse frequencies of the structure were evaluated to ensure that they meet the required dynamic criteria, as defined by TM 2.10.10. In the case of the vertical and torsional frequencies, two conditions were assessed: the first with upper bound mass and lower bound stiffness (Condition 1); the second with lower bound mass and upper bound stiffness (Condition 2). Condition 1 was also adopted for the transverse frequency analysis, as this generated the most onerous frequencies in this case.

The natural frequencies were found to be within the defined limits. See Table 4.4-16 for a summary of the results for the 510 ft continuous span. The effective span length used for frequency analysis was 170 ft as per TM2.10.10 Section 6.8.2. For limits and natural frequencies of other thermal spans, see the complete structural calculations. No torsional frequencies were found below 3.246 Hz for condition 1 and 3.428 Hz for condition 2.

Table 4.4-16
Kings/Tulare Regional Station Frequency Check Results

	Vertical Frequency (Hz)	Torsional Frequency (Hz)	Transverse Frequency (Hz)
Lower Limit	1.950	3.246 condition 1 3.428 condition 2	1.2
Upper Limit	4.064	N/A	N/A
Condition 1	2.705	greater than 3.246	greater than 1.2
Condition 2	2.857	greater than 3.428	N/A

It has been found that the structure complies with the vertical frequency requirements of TM 2.10.10 with a reasonable margin. The section sizes specified are therefore larger than would be required and may be further optimized by the contractor.

As the vertical frequencies are governed in large part by the ground conditions, access to site-specific GI data may reveal more beneficial soil parameters, and permit further savings in the refinement of the design.

4.4.8 Rail Serviceability and Track-Structure Interaction Results

The model spans were analyzed for deflections and rail stresses and evaluated against the limits prescribed in TM 2.10.10. This included the assessment of global deflections of the structure, the relative rotations and displacements at the rails and expansion joints, and the relative twist of the deck.

Several MIDAS models were developed to model train loads at the most onerous locations on or immediately adjacent to, the most critical joints in the structure. The various train locations were coupled with the load permutations and cases specified in TM 2.10.10 to envelope the worst deflections in the structure.

Group 1c applies to the Kings/Tulare Regional Station structure where the structure supports more than two tracks. For the continuous span of interest between bents 44 and 47, for which worst case results are presented in the following tables, three tracks are loaded in Group 1c: the two through tracks and one exterior storage track. Only vertical live load (no lateral live load) was applied to the storage track for Group 1c. When applying load combinations for OBE events, only half the number of total tracks shall be considered simultaneously (TM 2.10.4). Therefore, a maximum of two tracks were loaded for load Groups 3 and 5.

It has been demonstrated from the analysis that the spans meet most of the requirements of TM 2.10.10 except for the limit of rotation about transverse axis of the bridge. For case specific results at various bent locations, see the structural calculations report. See Table 4.4-17 to Table 4.4-23 for a summary of the results at the most onerous locations.

Table 4.4-17
Kings/Tulare Regional Station Track Serviceability Results (1)

Group	**Vertical Deflection** L = 230 ft		**Transverse Deflection** L = 230 ft	
	Limit	**Kings/Tulare**	**Limit**	**Kings/Tulare**
Group 1a	1.090	0.780	0.734	0.017
Group 1b	1.588	0.839	1.419	0.033
Group 1c	4.600	1.009	2.293	0.060
Group 3	N/A	N/A	2.293	0.401

Table 4.4-18
Kings/Tulare Regional Station Track Serviceability Results (2)

Group	Rotation about Vertical Axis		Rotation about Transverse Axis	
	Limit	Kings/Tulare	Limit	Kings/Tulare
Group 1a	0.0007	0.0000	0.0012	0.0003
Group 1b	0.0010	0.0000	0.0017	0.0002
Group 3	0.0021	0.0001	0.0026	0.0002

Table 4.4-19
Kings/Tulare Regional Station Track Serviceability Results (3)

Group	Deck Twist	
	Limit	Kings/Tulare
Group 1a	0.06	0.018
Group 1b	0.06	0.056
Group 3	0.17	0.099

Table 4.4-20
Kings/Tulare Regional Station Track Structure Interaction Results (1)

Group	Relative Longitudinal Displacement	
	Limit L = 510ft	Kings/Tulare
Group 4	2.2240	0.76689
Group 5	2.9420	0.41594

Table 4.4-21
Kings/Tulare Regional Station Track Structure Interaction Results (2)

Group	Relative Vertical Displacement	
	Limit	Kings/Tulare
Group 4	0.25	0.0541
Group 5	0.50	0.0213

Table 4.4-22
Kings/Tulare Regional Station Track Structure Interaction Results (3)

Group	Relative Transverse Displacement	
	Limit	Kings/Tulare
Group 4	0.08	0.00899
Group 5	0.16	0.0242

Table 4.4-23
Kings/Tulare Regional Station Track Structure Interaction Results (4)

Group	Permissible Axial Rail Stress	
	Limit	Kings/Tulare
Group 4	±14	-6.2 to 5.8
Group 5	±23	-2.7 to 10.1

4.4.9 Force Results

The key components of the station structure have been checked for structural adequacy to assess the validity of the section sizing. In addition to the station structure, columns supporting the spans of the typical viaduct were also checked.

The force checks comprised of the RC section design of the columns and pile caps and the RC design of the piled foundations.

Table 4.4-24
Column Strength Check – Load Case, Axial, and Flexural

	Strength 1	Strength 5
Axial Demand (k)	5,736	3,205
Shear SRSS (k)	1,412	1,329
Moment M3 Demand (k-in)	363,308	295,723
Moment M2 Demand (k-in)	82,572	176,139
Moment SRSS Demand/Capacity Ratio	0.63	0.671

Table 4.4-25
Pile Strength Check – Governing Axial/Moment Load per Pile

	Strength 1	Strength 5
Axial Demand (k)	1434	801
Shear Demand SRSS	647	598
Flexural Demand (k-in)	70,654	65,274
Demand/Capacity Ratio	0.20	0.19

4.4.10 Seismic Results

4.4.10.1 Columns

The displacement capacity and displacement ductility demand of the columns were assessed in accordance with the CSDC. One percent reinforcement ratio is provided for each column. The reinforcing steel detail is shown in Figure 4.4-3. The assessment was completed using a combination of global and local SAP models and running pushover analyses in both the X and Y directions.

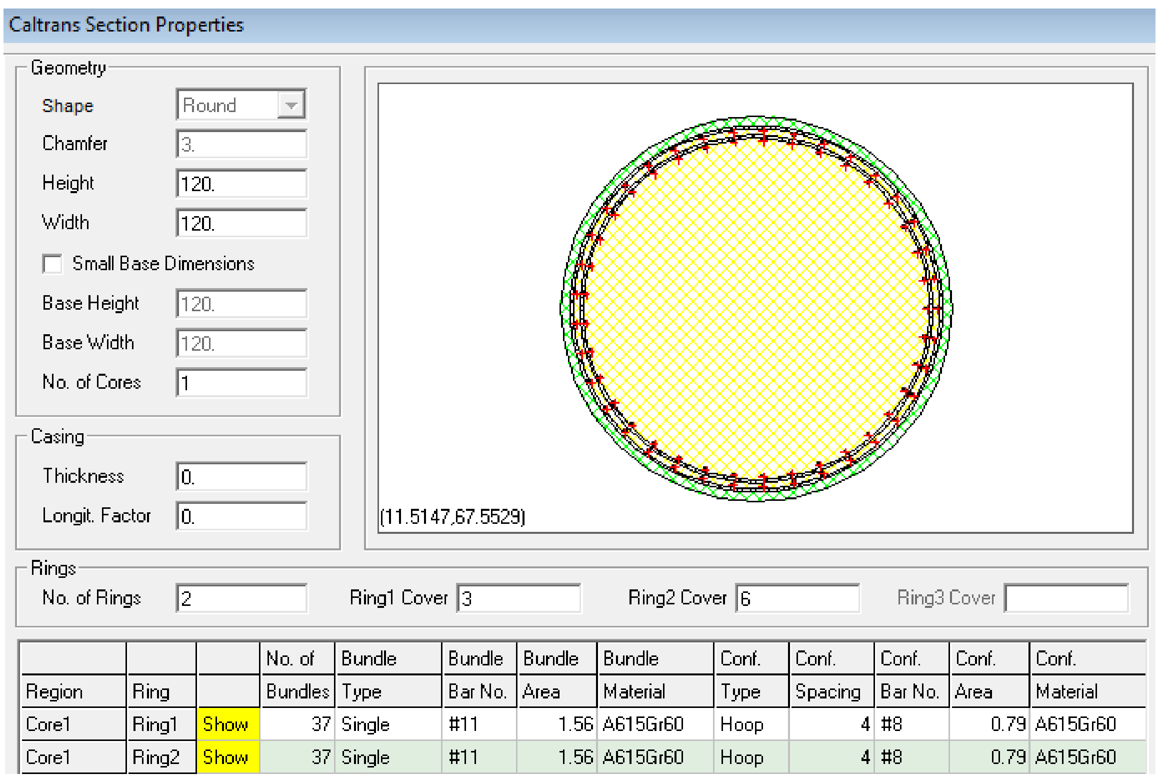

Figure 4.4-3
Kings/Tulare Regional Station 10ft Column Design

It has been demonstrated from the analysis and calculations that the Kings Tulare Regional Station concept is structurally viable and meets the requirements of the CSDC. See Table 4.4-26 to Table 4.4-29 for a summary of the results.

Table 4.4-26
Seismic Displacement Check – Displacement Demand Ductility Check

	Displacement Ductility Upper Limit	MCE Displacement ΔD (in)	Yield Displacement ΔY (in)	Displacement Ductility, μD
Longitudinal	5	1.165	0.2659	4.381
Transverse	5	1.206	0.2956	4.079

Table 4.4-27
Seismic Displacement Check – Capacity Ductility Check

	Capacity Ductility Lower Limit	Yield Column Displacement $\Delta YCOL$ (in)	Collapse Column Displacement $\Delta cCOL$ (in)	Capacity Ductility, μC
Longitudinal	3	0.351	4.455	12.7
Transverse	3	0.319	4.423	13.9

Table 4.4-28
Seismic Displacement Check – Displacement Demand/Capacity Ratio

	MCE Displacement Demand ΔD (in)	Displacement Capacity Δc (in)	Demand/Capacity
Longitudinal	1.165	4.721	0.25
Transverse	1.206	4.719	0.26

Table 4.4-29
Column Strength Check

	Envelope
Plastic Moment, Mp (k-in)_	641,479
Overstrength Moment, Mo (k-in)	769,775
Overstrength Shear Demand, V (kips)	1,697
Shear Capacity ØVn (kips)	6,910

RFP No.: 13-57 – Addendum No. 2 - 06/30/2014

4.4.10.2 Foundation Design

The demand forces to be used for the foundation design are specified in the CSDC and include the service level moments, shears, axial loads and the moment demand induced by the column plastic hinging mechanism. The moment derived from the plastic hinging mechanism is taken as the plastic moment of the column, multiplied by an overstrength factor of 1.2 to give the overstrength moment.

The adoption of the overstrength factor is based upon the seismic design philosophy whereby the columns will always yield at the MCE event to protect the pile. The factor is required to account for risk that the column may develop a greater plastic moment capacity than the idealized values used in the design.

This approach has also been used for the design of the piled foundations of the nonstandard columns immediately adjacent to the crossover structure. An axial-moment diagram from the section designer module in CSiBridge was used to check the reinforcement design of the pile. A reinforcement ratio of 2% and #8 at 12-inch tie spacing was used for the design for 6.5ft diameter pile. Pile shear and moment demand derived from column overstrength moment are checked against capacity.

Table 4.4-30
Pile Capacity Protection – Column Overstrenth Demand/Capacity Ratio per pile

	Max Axial	Min Axial
Extreme 3 Axial Demand (k)	1,697	854
Overstrength Moment Demand (k-in)	186,083	157,184
Momend Capacity (k-in)	199,735	185,754
Moment D/C	0.93	0.85
Overstrength Design Shear Demand (k)	1,692	1,429
Sehar Capacity (k)	2,676	2,676
Shear D/C	0.63	0.53

RFP No.: 13-57 – Addendum No. 2 - 06/30/2014

4.5 Limits of Standard Bridge Design and Special Bridge Design

It is assumed that the standard bridge design is suitable for use on spans 1 to 13 and 63 to 86. Spans 14 to 62 are considered in this design.

4.6 Construction Methods Assessment

The Kings –Tulare Regional Station slab is located in a largely green field site with access to the local road network. The structure is assumed to be in situ post tensioned concrete which is the preferred structural form for highway structures in California and so presents no unusual difficulties for the local contractors. The assumed construction methodology for the simply supported spans and also for some of the continuous spans is to deck out the soffit area of the structure using falsework and cast the superstructure in situ, span by span and also with at least 2 stages of concreting (soffit slab and webs followed by deck slab).

For the spans crossing SR 198 and the Cross-Valley Railroad, it is possible that the superstructure could be constructed in situ with suitable agreement for temporary operation from Caltrans and the CVRR. However, it is also possible that these spans could be constructed as either precast beam elements with longitudinal connections or as segmental precast balanced cantilever construction.

None of these erection methods excludes other methods of construction from consideration. It is likely that contractors will prefer to use methods that they have used successfully in the past. The assessment described here therefore represents a subset of methods that could be used.

4.7 Temporary Construction Loadings Considered

No specific loadings have been considered for the temporary stages described.

4.8 Temporary Construction Easements

A general temporary construction easement of 15 feet width outside of the permanent right-of-way has been indicated for the full length of the station area. In the area of Ponderosa Road the boundary of the right-of-way has been set at the western edge of the roadway. This is within 5 feet of the edge of the structure overhead and so the constraint of working within this width will need to be managed by the contractor and future operators. It is possible that a temporary construction easement can be agreed to facilitate construction, but it is likely that there will be a requirement to maintain access to Ponderosa Road throughout the construction period.

4.9 Traffic or Pedestrian Diversion and Control

Apart from SR 198, the location of these spans is in a very rural and lightly trafficked location. It is not expected that elaborate measures for control of traffic or pedestrians will be required. Access to Ponderosa Road is likely to be a major concern for the residents as they would be completely cut off is the road was blocked. It is therefore likely that the contractor will be required to maintain access to ponderosa Road at all times during construction.

Work over and adjacent to SR 198 will be subject to the agreement of Caltrans and is likely to include restrictions on working over the live roadway and requirements for lane closure and traffic diversion during key stages of the construction.

Should HSR ridership justify the construction of the KTR station, the required works would mostly take place under the viaduct structure. As currently envisaged, the only exception to this would be the construction of the station platforms. These could either be constructed in-situ or precast off site. This has not been considered further in this report.

4.10 Drainage Concept

The track drainage for the Kings/Tulare Regional Station structure will be carried from deck level through to a permanent drainpipe fitted within the voids of the concrete deck girders. This pipe will be connected to downpipes cast into the columns. The downpipes will outfall near ground level to the surface drainage system.

4.11 Emergency Egress and Escape Provision

Provision for emergency access will be made in accordance with TM 2.8.1Safety and Security Design Requirements R0 (March 12th, 2012), TM 2.3.3 HST Aerial Structure R0 (June 2nd, 2009), NFPA130 and NFPA101. Emergency access points are required at maximum 2,500-foot intervals along aerial structures with access stairs to be located every 2.5 miles. It is also a requirement that access to the trackside is provided at each systems site which are also at approximately 2.5 mile intervals. Therefore, access stairs have been provided at each systems site and emergency ladder access turnarounds are provided at 2,500-foot nominal centers between systems sites. The Kings/Tulare Regional Station is not of sufficient length to require emergency access stairs.

Table 4.11-1
Escape Stair Locations

STA	Locale	Egress features
1919+00	Adjacent to Grangeville Boulevard	Ladder Access Turnaround
1947+00	Adjacent to station platform area	Ladder Access Turnaround
1978+00*	Adjacent to SR 198	Access stairs to trackside for systems site†
1960+00*	Near cross valley railroad	Access stairs to trackside for systems site†
1938+00*	Between Grangeville Boulevard and proposed station structure	Access stairs to trackside for systems site†
1931+00*	Between Grangeville Boulevard and proposed station structure	Access stairs to trackside for systems site†
1923+00*	Adjacent to Grangeville Boulevard	Access stairs to trackside for systems site†

Notes.
* Two alternative locations for systems sites are indicated on the drawings.
†Access via the station structure may be better alternative for providing access to the trackside than dedicated stairs in these locations.

4.12 Inspection, Service, and Maintenance Access

The Kings/Tulare Regional Station superstructure will be a simple multicellular concrete section that is inspectable from both inside and outside.

4.13 Utilities Affected and Disposition

The major utilities that cross the route of the viaduct are identified on the composite utility drawings.

4.14 Noise Mitigation and Acoustic Treatment

No specific features have been included in the structure to mitigate the noise generated by the passage of trains. The project environmental impact statement (EIS) will define what measures are required to mitigate noise impacts. In the station area there will be parapets to the rear of the platforms (when constructed) which may assist in providing some acoustic protection and which can be extended or enhanced to provide other protection as necessary.

The structure is considered sufficiently robust to accommodate the addition of, for example, noise protection fencing, should this be required for mitigation of impact.

4.15 Compliance with System-wide Bridge Aesthetics Features

TM 200.06, "Aesthetic Guidelines for Non-Station Structures" provides guidance on the appearance targets for the CHSTP. The scheme detailed on the PE4P drawings and analyzed represents the functional baseline case on which the DB contractors are encouraged to improve in discussion with the Authority.

4.16 Geotechnical Parameters Used for Design

The geotechnical parameters used for the analysis of this structure are described in the Geotechnical Design Report attached at Appendix A.

Section 5.0

Kaweah SR 43 Crossing

5.0 Kaweah SR 43 Crossing

The Kaweah SR 43 Crossing carries the HSR across a section of SR 43 that will be depressed to allow the HSR route to stay close to existing grade. The skew angle at which the two routes cross is substantially greater than permitted by TM 2.10.10 and so the HSR structure is designed to be square to the HSR alignment. This has the effect of increasing the individual spans to clear the SR 43 traveled way. Additionally, Caltrans has plans for future widening of SR 43 in this area and so the structure configuration spans over the space reserved for this future widening.

5.1 Structure Form

The widths of current and future SR 43 combined with the skew of the crossing dictate the use of a two-span truss structure. Each span is 280 feet and allowing for additional space for joints gives an overall structure length of 573 feet 6 inches, from abutment centerline to abutment centerline. As the approaches to the structure are at-grade, there are no approach spans at either end of the structure.

The center support takes the form of a single large diameter column supporting a hammerhead column cap which is arranged perpendicular to the HSR alignment. A single column has been used to allow the median width of the future SR 43 to be minimized.

A diagram of the structure is shown in Figure 5.1-1. The structure model used for this analysis is shown in Figure 5.1-2 below.

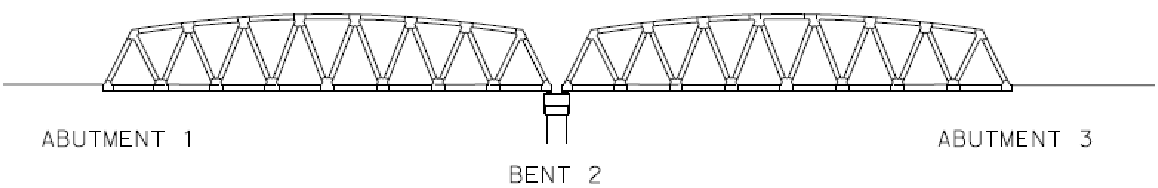

Figure 5.1-1
Kaweah SR 43 Crossing - Diagram

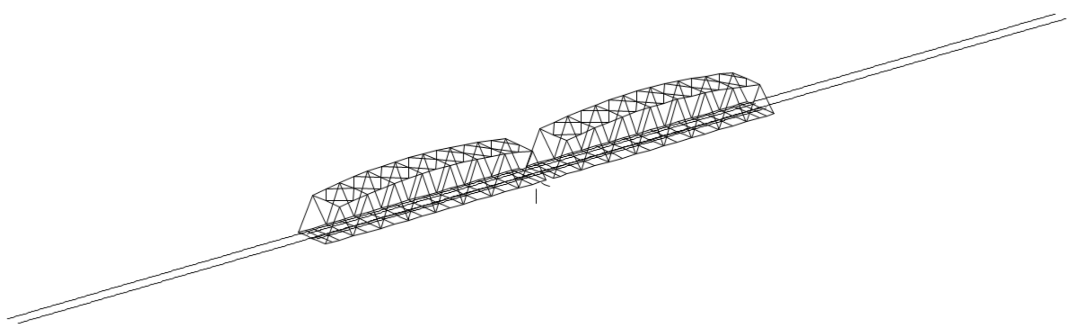

Figure 5.1-2
Kaweah SR 43 Crossing - SAP Model

The truss form follows the same overall style as used for CP1C. The top chord member for each span is a 3-by-3-foot steel box section. The truss depth increases at each node connection to a maximum of 45 feet between member centerlines. The structure depth is similar to the shape of the bending moment diagram so that chord stresses are relatively uniform throughout the span. A reinforced concrete deck slab spans between the floor beams. This acts compositely with the bottom chord and also the transverse floor beams.

The central column is supported on a pilecap with 6-No 6-foot-6-inch-diameter CIDH piles arranged to provide the maximum stiffness against forces that are longitudinal to the structure (i.e. thermal, traction and braking).

5.2 Structure Importance Classification

TM 2.3.2 paragraph 2.2.1 defines all structures supporting the high-speed tracks to be primary structures because they will be required to be reinstated to allow resumption of train service after an earthquake.

This classification implies the following:

- Design life is 100 years.
- Seismic design must comply with TM 2.10.4.
- When applying the AASHTO LRFD code, values for the importance, ductility, and redundancy factors, hI, hD, and hR have been chosen as follows:
  - Importance factor hI = 1.05.
  - Ductility factor hD = 1.05 for strength calculations.
  - Redundancy factor h_R = 1.05 for nonredundant elements, 1.0 otherwise.

5.3 Key Design Features and Site Constraints

5.3.1 Ground Conditions

Geotechnical advice is based on historic borehole records from Caltrans projects located in the vicinity of the route, as no local borehole data were available. The foundation spring stiffness has therefore been based upon the lower-bound interpretation of the soil parameters, using the nearest borehole data and engineering judgment. Detailed design will be based on investigation results which are expected to demonstrate that this approach is conservative.

See the Geotechnical Design Report in Appendix A for details.

5.4 Summary of Analysis and Results

All sections have been checked for resonance effects, rail serviceability and track-structure interaction limits, and force demands. In all cases the structure has been found to be satisfactory.

Based upon the calculations the preliminary designs are in full compliance with the TMs and are capable of being developed into an acceptable design solution.

The main results are summarized in Tables 5.4-1 to 5.4-8.

5.4.1 Modeling

Both SAP and CSiBridge modeling programs were used for the analysis of the Kaweah SR 43 Crossing. Several models of each section were required in order to represent the different conditions of the structure at different loading cases and for different design checks, in accordance with TM 2.10.4 and 2.10.10.

The truss members were represented by stick elements. Piles were represented by nonlinear springs, using equivalent stiffness values to correctly model the soil structure interaction based upon soil parameters in the GDR (Appendix A). The pile cap and pile group effects were modeled using rigid links connecting the top of the piles to the pier bent elements. In the case of the transverse frequency analysis, pinned restraints were added in place of the bearings, as only the flexibility of the superstructure is to be considered.

5.4.2 Frequency Results

The vertical, torsional, and transverse frequencies of the structure were evaluated to ensure that they meet the required dynamic criteria, as defined by TM 2.10.10. In the case of the vertical and torsional frequencies, two conditions were assessed: the first with upper bound mass and lower bound stiffness (Condition 1); the second with lower bound mass and upper bound stiffness (Condition 2). Condition 1 was also adopted for the transverse frequency analysis, as this generated the most onerous frequencies in this case.

The natural frequencies were found to be within the defined limits. The effective length used for frequency analysis is the truss span of 280-feet. See Table 5.4-1 for a summary of the results.

Table 5.4-1
Kaweah SR 43 Crossing Frequency Check Results

	Vertical Frequency (Hz)	Torsional Frequency (Hz)	Transverse Frequency (Hz)
Lower Limit	1.70	2.73 condition 1 2.88 condition 2	1.2
Upper Limit	3.41	N/A	N/A
Condition 1	2.28	4.667	2.45
Condition 2	2.40	4.960	N/A

5.4.3 Rail Serviceability and Track-Structure Interaction Results

The truss spans were analyzed for deflections and rail stresses and evaluated against the limits prescribed in TM 2.10.10. This included the assessment of global deflections of the structure, the relative rotations and displacements at the rails and expansion joints, and the relative twist of the deck.

Several SAP models were developed to model train loads at the most onerous locations on the structure. The various train locations were coupled with the load permutations and cases specified in TM 2.10.10 to envelope the worst deflections in the structure.

It has been demonstrated from the analysis that the truss spans meet all of the requirements of TM 2.10.10 and all rotations, deflections and rail stresses are with limits. The governing span length for deflection limits is 280-feet. See Tables 5.4-2 to 5.4-8 for a summary of the results. Results shown are for the worst cases only. Joint specific results can be found in the complete calculation report along with supporting calculations.

Table 5.4-2
Kaweah SR 43 Crossing Track Serviceability Results (1)

Group	Vertical Deflection		Transverse Deflection	
	Limit	Kaweah SR 43 Crossing	Limit	Kaweah SR 43 Crossing
Group 1a	1.627	0.641	1.088	0.195
Group 1b	2.366	1.287	2.104	0.000
Group 3	N/A	N/A	3.399	0.729

Table 5.4-3
Kaweah SR 43 Crossing Track Serviceability Results (2)

Group	Rotation about Vertical Axis (rad)		Rotation about Transverse Axis (rad)	
	Limit	Kaweah SR 43 Crossing	Limit	Kaweah SR 43 Crossing
Group 1a	0.0007	0.00017	0.0012	0.0008
Group 1b	0.0010	0.00003	0.0017	0.00134
Group 2	0.0021	0.00017	0.0026	0.0008
Group 3	0.0021	0.00057	0.0026	0.0011

Table 5.4-4
Kaweah SR 43 Crossing Track Serviceability Results (3)

Group	Deck Twist (Rad)	
	Limit	Kaweah SR 43 Crossing
Group 1a	0.0011	0.00019
Group 1b	0.0011	0.00020
Group 3	0.0030	0.00026

Table 5.4-5
Kaweah SR 43 Crossing Track Structure Interaction Results (1)

Group	Relative Longitudinal Displacement at Expansion Joints (in)	
	Limit	Kaweah SR 43 Crossing
Group 4	1.807	1.42
Group 5	2.733	0.934

Table 5.4-6
Kaweah SR 43 Crossing Track Structure Interaction Results (2)

Group	Relative Vertical Displacement (in)	
	Limit	Kaweah SR 43 Crossing
Group 4	0.25	0.073
Group 5	0.50	0.071

Table 5.4-7
Kaweah SR 43 Crossing Track Structure Interaction Results (3)

Group	Relative Transverse Displacement (in)	
	Limit	Kaweah SR 43 Crossing
Group 4	0.08	0.003
Group 5	0.16	0.020

Table 5.4-8
Kaweah SR 43 Crossing Track Structure Interaction Results (4)

Group	Permissible Axial Rail Stress (ksi)	
	Limit	Kaweah SR 43 Crossing
Group 4	±14	12.0 to –8.6
Group 5	±23	22.6 to –18.7

5.4.4 Force Results

The key components of the Kaweah SR 43 Crossing have been checked for structural adequacy to assess the validity of the section sizing. The force checks comprised of the RC section design of the columns and pile caps and the RC design of the piled foundations. Strength check results for Bent 2and Abutment 3 are summarized in Tables 5.4-9.

Table 5.4-9
Column Strength Check – Load Case, Axial, and Flexural

	Bent 2*		Abut 3*
Strength Load Combination	1	5	1‡
Axial Demand (k)	16,321	11,733	21,483
Moment M3 Demand (k-in)	1,163,344	314,258	1,212,553
Moment M2 Demand (k-in)	2,058	46,861	3,008
Moment SRSS Demand/Capacity Ratio	0.557	0.309	0.597

* For location of Bent 2 and Abut 3, see Figure 5.1-1.
‡ Strength 1 is reported for Abutment 3, strength 5 will be checked for Abutment 3 under OBE seismic in detailed design.

Table 5.4-10
Pile Strength Check – Governing Axial/Moment Load per Pile

	Governing Pile: Strength 5
Governing Axial Demand (k)	2,354
Flexural Demand (k-in)	18,452
Demand/Capacity Ratio	0.126

5.4.5 Seismic Results

5.4.5.1 Columns

A pushover analysis was carried out on the structure of the Kaweah SR 43 Crossing including the single column pier in the median of the future SR 43. The displacement capacity and displacement ductility demand of the column was assessed in accordance with the CSDC. 1.2% reinforcement ratio is provided for the 15-ft diameter column. The reinforcing steel detail is shown in Figure 5.4-1. The assessment was completed using a combination of global and local SAP models with pushover analyses in both the X and Y directions.

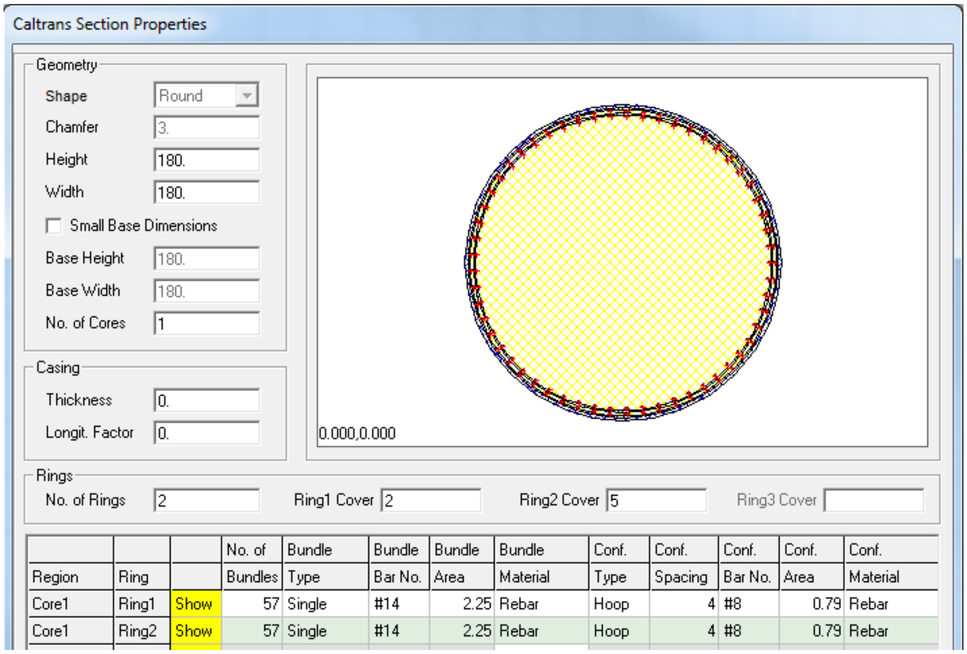

Figure 5.4-1
Section Properties – Kaweah SR 43 Crossing 15-foot Column

It has been demonstrated from the analysis and calculations that the Kaweah SR 43 Crossing is structurally viable and meets the requirements of the CSDC. See tables 5.4-9 to 5.4-12 for a summary of the results.

Table 5.4-11
Seismic Displacement Check – Displacement Demand Ductility Check

	Displacement Ductility Upper Limit	MCE Displacement ΔD (in)	Yield Displacement ΔY (in)	Displacement Ductility, μD
Longitudinal	5	0.3401	1.3574	0.2506
Transverse	5	1.0108	1.7131	0.5901

Table 5.4-12
Seismic Displacement Check – Capacity Ductility Check

	Capacity Ductility Lower Limit	Yield Column Displacement ΔYCOL (in)	Collapse Column Displacement ΔcCOL (in)	Capacity Ductility, μC
Longitudinal	3	0.102	0.642	6.27
Transverse	3	0.099	0.642	6.48

Table 5.4-13
Seismic Displacement Check – Displacement Demand/Capacity Ratio Check

	MCE Displacement Demand ΔD (in)	Displacement Capacity Δc (in)	Demand/Capacity
Longitudinal	0.340	1.999	0.17
Transverse	1.011	2.355	0.43

Table 5.4-14
Column Strength Check

	Governing Bent
Plastic Moment, Mp (k-in)	1,721,340
Overstrength Moment, Mo (k-in)	2,065,608
Overstrength Shear Demand, V (kips)	4,848
Shear Capacity ØVn (kips)	7,368

5.4.5.2 Foundations

The demand forces to be used for the foundation design are specified in the CSDC and include the service level moments, shears, axial loads and the moment demand induced by the column plastic hinging mechanism. The moment derived from the plastic hinging mechanism is taken as the plastic moment of the column, multiplied by an overstrength factor of 1.2 to give the overstrength moment demand. An axial-moment diagram from the section designer module in CSiBridge was used to check the reinforcement design of the pile. A minimal reinforcement ratio of 2% and #8 at 12-inch tie spacing was used for 6.5ft diameter pile. Pile shear and moment demand derived from column overstrength moment are checked against capacity.

The adoption of the overstrength factor is based upon the seismic design philosophy whereby the columns will always yield at the MCE event to protect the pile. The factor is required to account for risk that the column may develop a greater plastic moment capacity than the idealized values used in the design.

Table 5.4-15
Pile Strength Check – Column Overstrenth Demand per Pile

	Bent 2
Extreme 3 Axial Demand (k)	3,225
Overstrength Moment Demand (k-in)	92,050
Moment Capacity (k-in)	239,000
Moment D/C	0.385
Overstrength Design Shear Demand (k)	1023
Shear Capacity (k)	2676
Shear D/C	0.382

5.5 Limits of Standard Bridge Design and Special Bridge Design

The entire structure is considered to be Nonstandard as defined by TM 2.10.10.

5.6 Construction Methods Assessment

This structure is to be located in the middle of a cutting that must be excavated to provide the new roadway for the existing SR 43. It is anticipated that this would be accomplished by constructing a temporary diversion of SR 43 at-grade while the cutting is excavated. The traffic would then be diverted onto the newly constructed section. While this work is underway it would be likely that the foundations for the HSR structure would be constructed.

It is assumed that the most efficient operation would be to excavate the entire width of cutting in one operation and, to avoid conflict, it would then be logical to locate the temporary diversion behind one of the proposed abutments of the HSR structure.

Once the supports are constructed it would be most likely that the superstructure for both spans would be constructed by erecting the steelwork and deck slab supported on falsework. Traffic could be redirected onto the newly reconstructed roadway either after completion or prior to final fit out of the new superstructure.

It is likely that contractors will have their own preferred methods of construction for structures of this type, but the assessment described here represents one of many methods that could be used.

5.7 Temporary Construction Loadings Considered

No specific loadings have been considered for the temporary stages described.

5.8 Temporary Construction Easements

A general temporary construction easement of 15 feet width outside of the permanent right-of-way has been indicated for the full length of the viaduct. This should be sufficient for the foreseeable requirements for construction of such a structure.

5.9 Traffic or Pedestrian Diversion and Control

SR 43 is a major state facility and is likely to be highly trafficked at all times. To facilitate construction it would be necessary to divert the roadway as it would be impracticable to excavate the cutting any other way. SR 43 is unlikely to have a significant pedestrian flow as it is in a rural area. The temporary traffic diversion should however also be capable of providing a safe route for pedestrians in emergency situations. Any temporary diversion would be subject to the agreement of Caltrans.

5.10 Drainage Concept

The drainage of the structure will be collected at deck slab level with valley gutter between the two tracks, and carried longitudinally in pipes to the ends of the deck at each abutment. The runoff will be conveyed in pipes down the sides of the abutments to infiltration swales or other BMPs within the HST right-of-way as proposed under Section 2, Stormwater Quality Management Report for the Fresno to Bakersfield Section of California High-Speed Train Project Record Set 15% Design Submission, dated December 2013.

5.11 Emergency Access Provision

This bridge is a short interruption in a substantial length of at-grade route and is not of sufficient size to warrant special emergency access provisions.

5.12 Inspection, Service, and Maintenance Access

The main truss members will have steel truss girder spans which are largely composed of hollow box sections. It is expected that these box sections will be accessible through access covers at key locations for inspection and maintenance purposes. If the specification permits, it is also possible that the DB Contractor may propose that the box sections are fully sealed to prevent moisture ingress, in which case the internal surfaces will not be inspectable without additional work to break and repair the seals. The external surfaces of the truss members will are accessible for inspection using a hydraulic access platform subject to suitable safety procedures for working at the trackside and also for working over SR 43.

It will be necessary to agree appropriate traffic control and or road closures with Caltrans to facilitate such inspections.

5.13 Utilities Affected and Disposition

This structure is located in a rural area which is remote from existing utility corridors. There are likely to be some longitudinal utilities associated with SR 43 and these should be diverted with the roadway to enable the construction of the cutting.

5.14 Noise Mitigation and Acoustic Treatment

No specific features have been included in the structure to mitigate the noise generated by the passage of trains. The project environmental impact statement (EIS) will define what measures are required to mitigate noise impacts.

However, this structure is considered sufficiently robust to accommodate the addition of, for example, noise protection fencing, should this be required for mitigation of impact.

5.15 Compliance with System-wide Bridge Aesthetics Features

TM 200.06, "Aesthetic Guidelines for Non-Station Structures" provides guidance on the appearance targets for the CHSTP. The scheme detailed on the PE4P drawings and analyzed represents the functional baseline case on which the DB contractors are encouraged to improve in discussion with the Authority.

5.16 Geotechnical Parameters Used for Design

The geotechnical parameters are described in the Geotechnical Design Report attached at Appendix A.

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Section 6.0

Corcoran Crossover Structure

6.0 Corcoran Crossover Structure

The Corcoran Viaduct is 5,666 feet in length and composed of three sections. The north and south sections are standard viaduct. The middle section is a crossing of SR 43 and the BNSF tracks and named as Corcoran Crossover Structure. The crossover structure is considered to be nonstandard and complex and is the subject of this analysis.

6.1 Structure Form

For this analysis, the Corcoran Crossover Structure is conceived as a 2,426-foot-long elevated slab, supported on multiple columns to either side of the SR 43 and BNSF routes. Each 6-foot-diameter column is positioned at nominal 30-foot centers along the length of the structure and is founded on a single 9-foot-diameter drilled shaft pile of 170-foot depth. The middle section is a two-span structure which crosses both SR 43 and the BNSF corridor. Due to the skew of the crossing the structure extends to the north as a single span crossing of SR 43 only and to the south as a single span crossing of the BNSF only. This gives the plan of the structure Z like appearance as can be seen from the structure model in Figure 6.4-1.

The slab section is constructed from 6-foot deep, precast, PC beams supported on a 12-foot deep in situ concrete column cap beam which runs parallel to the railway. The beam spans are approximately perpendicular to the SR 43 and BNSF corridor and are placed immediately adjacent to one-another, typically this gives a beam spacing of 4 feet on centers. The deck slab is 6 inches in thickness and is intended to act compositely with the beams. The superstructure has been divided into individual thermal units of approximately 150 to 200 feet in length to reduce the thermal displacement and force effects. Movement between adjacent thermal units of the slab is controlled by dowelled connections, which allow relative longitudinal and vertical displacements but not relative transverse displacement. A similar dowelled connection is provided between the end panel of the slab and the adjacent span of the standard viaduct.

The standard spans of the viaduct are formed from precast, prestressed PC box girders and are seated upon RC columns, which are in turn each founded on a pile cap with a group of 4no. 6-foot-6-inch diameter drilled shaft piles. Due to clearance constraints near to the BNSF right-of-way and reduced loading, the columns immediately adjacent to the crossover structure modify the general foundation arrangement by using a two-pile group with a narrower pile cap.

6.2 Structure Importance Classification

TM 2.3.2 paragraph 2.2.1 defines all structures supporting the high-speed tracks to be primary structures because they will be required to be reinstated to allow resumption of train service after an earthquake.

This classification implies the following:

- Design life is 100 years.
- Seismic design must comply with TM 2.10.4.
- When applying the AASHTO LRFD code, values for the importance, ductility, and redundancy factors, hI, hD, and hR have been chosen as follows:
  - Importance factor hI = 1.05.
  - Ductility factor hD = 1.05 for strength calculations.
  - Redundancy factor h_R = 1.05 for nonredundant elements, 1.0 otherwise.

6.3 Key Design Features and Site Constraints

6.3.1 Dowel Connections

Dowel connections are located at the breaks between adjacent thermal units of the deck slab and at the interface connections between the crossover structure and the standard viaduct sections. The purpose of the dowels is to control the relative movement between the thermal units, and in particular the movement at the rails. The dowels are aligned to be parallel with the rail axes at the interface between the units to ensure that the relative structure movement is also along the rail axis. This ensures that lateral distortions are minimized. The dowels are assumed to allow relative rotation about the transverse axis and displacement in the longitudinal and vertical directions, but limit all other degrees of freedom.

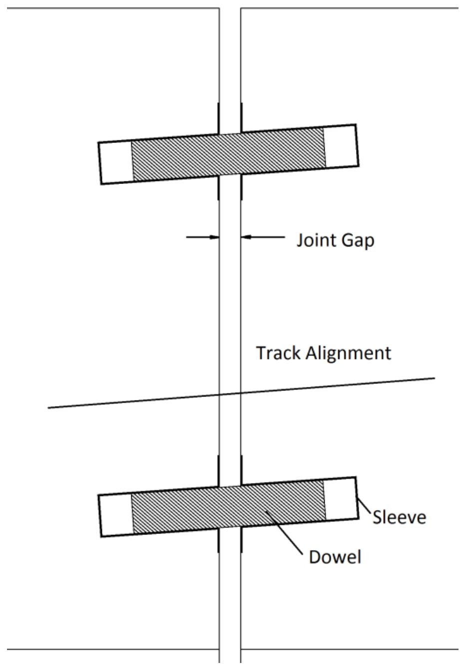

Figure 6.3-1
Detail of Dowels in One Column Cap Beam

As the dowels are aligned with the rails, expansion joints between the adjacent thermal units are not required to be perpendicular to the rail and are not in this case. It has instead been assumed that the joints will be aligned parallel to the cross beams. This requires the joint design to consider a minor component of lateral displacement with longitudinal displacement, but this is considered to be within the capability of typically available structure joints. Alternatively, for the simplification of track clip arrangement the joint could be made to be perpendicular to the rail in the vicinity of the HSR tracks and revert to being parallel to the beams outside this area.

The merits of these variations should be investigated further during the design development stage.

6.3.2 Ground Conditions

The geotechnical parameters used for the analysis are based on historic borehole records from Caltrans projects located in the vicinity of the route, as no project-specific or local borehole data were available. The foundation spring stiffness has therefore been based upon the lower-bound interpretation of the soil parameters, using the nearest borehole data and engineering judgment. Detailed design will be based on investigation results which are expected to demonstrate that this approach is conservative.

See the GDR in Appendix A for details of parameters and spring stiffness used in the analysis.

6.3.3 BNSF Future Provision

Double tracking is planned by the BNSF for several locations between Port Chicago and Bakersfield. It is understood that the BNSF have no plans to install additional tracks in locations where double tracking is already provided. However, the BNSF require the HSR structure to clear span their operational right of way which is nominally 100 feet wide centered on the original track. At the location of the crossing, the current BNSF route is single track and so the structure makes provision for tracks to be constructed to the east and west should the BNSF require this in future.

6.4 Summary of Analysis and Results

The PC box girder spans on either side of the SR 43/BNSF Crossover Structure are classified as standard structures and do not fall within the scope of this preliminary design. The standard girders have been modeled, where necessary, in accordance with TM2.10.4, to ensure that the behavior of the crossover structure is fully representative.

All sections have been checked for resonance effects, rail serviceability and track-structure interaction limits, and force demands. In all cases the structure has been found to be satisfactory.

Based upon the calculations thus far it appears that the preliminary designs are in full compliance with the TMs and are capable of being developed into a fully compliant design solution.

The main results are summarized in Tables 6.4-1 to 6.4-12

6.4.1 Modeling

Both SAP and CSiBridge modeling programs were used for the analysis of the Corcoran Viaduct. Several models of each section were required in order to represent the different conditions of the structure for different loading cases and for different design checks, in accordance with TM 2.10.4 and 2.10.10.

The structural columns, cross beams, rails and RC girders were represented by stick elements. Piles were represented by nonlinear springs, using equivalent stiffness values to correctly model the soil structure interaction based upon soil parameters in the GDR (Appendix A). The pile cap and pile group effects were modeled using rigid links connecting the top of the piles to the column elements. All viaduct spans were connected to the bent cap elements with linear bearing springs, with the bridge articulation represented by either pinned or rolling spring properties. In the unique case of the transverse frequency analysis, pinned restraints were added in place of the bearings, as only the flexibility of the superstructure is to be considered. Note that for the type of structure under consideration, the fixity requirement of TM 2.10.10 fully restrains the superstructure from all transverse movement.

Foundation arrangements for the standard spans were those used in the Authority's representative's design for the standard viaducts and have been used accordingly in the structural models. These foundations have been checked using LPILE and Pilset and found to have adequate capacity.

Linear and nonlinear springs were used to represent boundary conditions and stiffness in the model. Non-linear boundary springs were used to model the non-linear behavior of rail clips and pile foundations. However, when running linear analyses such as model analysis and response spectrum analysis, these springs are assumed to operate in the linear stiffness range and are, therefore, modeled as linear boundary springs. In accordance with TM 2.10.10, upper and lower bound stiffness were taken into account as were upper and lower bound mass.

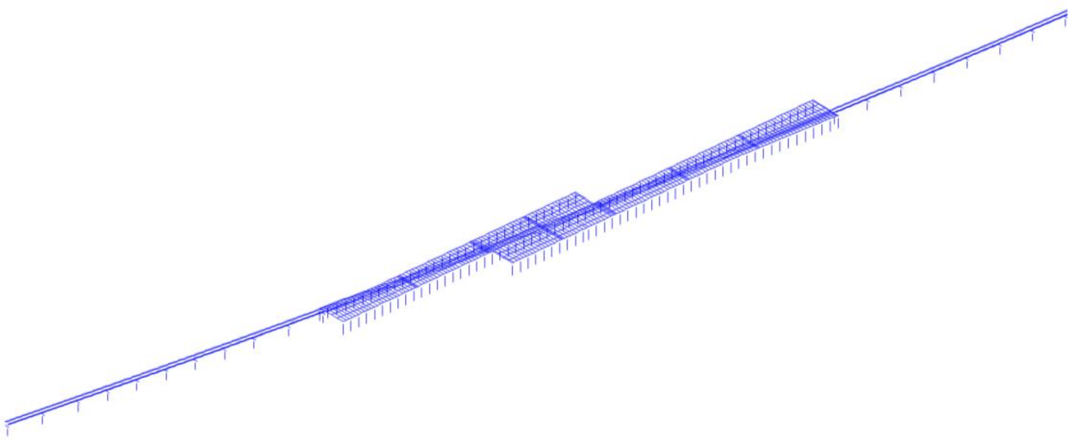

Figure 6.4-1
SAP Model

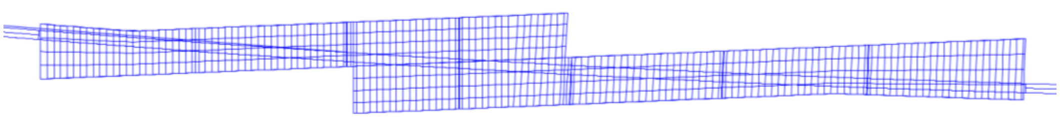

Figure 6.4-2
SAP Model - Plan View

6.4.2 Frequency Results

The vertical, torsional, and transverse frequencies of the structure were evaluated to ensure that they meet the required dynamic criteria, as defined by TM 2.10.10. In the case of the vertical and torsional frequencies, two conditions were assessed: the first with upper bound mass and lower bound stiffness (Condition 1); the second with lower bound mass and upper bound stiffness (Condition 2). Condition 1 was also adopted for the transverse frequency analysis, as this generated the most onerous frequencies.

In all thermal units of the crossover structure, the natural frequencies were found to be within the defined limits. See Table 4.4-1 for a summary of the results for the most onerous 115 feet transverse span with effective span length used for frequency analysis was 80.5-feet, and 100feet transverse span with effective span length of 73.82. For limits and natural frequencies of other thermal spans, see the complete structural calculations. No torsional frequencies were found below 4.36 Hz for condition 1 and 5.22 Hz for condition 2.

Table 6.4-1
Corcoran Crossover Structure Frequency Check Results

	Vertical Frequency L=100 ft (Hz)	Vertical Frequency L=115 ft (Hz)	Torsional Frequency L=100 ft (Hz)	Torsional Frequency L=115 ft (Hz)	Transverse Frequency (Hz)
Lower Limit	3.73	3.54	5.36 condition 1 6.33 condition 2	4.36condition 1 5.22 condition 2	1.2
Upper Limit	9.27	8.64	N/A	N/A	N/A
Condition 1	4.46	3.63	greater than 5.36	greater than 4.36	7.339
Condition 2	5.27	4.354	greater than 6.33	greater than 5.22	N/A

The fundamental frequency in the vertical direction is first observed in the modal results at the ends of the crossover structures, where the spans are at their longest due to the tapered geometry in plan. It has been found that the frequency in this direction is sensitive to the stiffness provided by the cross-beams and column sections, but also that it is particularly sensitive to the vertical stiffness of the foundations. Due to the soft soil case that has been considered in the design, the frequencies found are therefore also conservative.

It has been found that vertical frequency requirements govern the section dimensions of the crossover structure, with deeper sections needed to provide sufficient stiffness in order to satisfy TM 2.10.10. The section sizes specified are therefore larger than would be required from consideration of other effects such as strength.

As the vertical frequencies are governed in large part by the ground conditions, access to site-specific GI data may reveal more beneficial soil parameters, and permit savings with the refinement of the design. This can be investigated in further development of the design.

6.4.3 Rail Serviceability and Track-Structure Interaction Results

The crossover structure was analyzed for deflections and rail stresses and evaluated against the limits prescribed in TM 2.10.10. This included the assessment of global deflections of the structure, the relative rotations and displacements at the rails and expansion joints, and the relative twist of the deck.

Several SAP models were developed to model train loads at the most onerous locations on or immediately adjacent to, the crossover structure. The various train locations were coupled with the load permutations and cases specified in TM 2.10.10 to envelope the worst deflections in the structure.

It has been demonstrated from the analysis that the Corcoran Crossover structure meets all of the requirements of TM 2.10.10 and that all rotations, deflections and rail stresses are with limits. See Tables 6.4-2 to 6.4-8 for a summary of the results. Results shown are for the worst cases only. Joint specific results can be found in the complete calculation report along with supporting calculations.

Table 6.4-2
Corcoran Crossover Structure Track Serviceability Results (1)

Group	Vertical Deflection (in) L =106 ft		Transverse Deflection (in) L = 286 ft	
	Limit	Corcoran	Limit	Corcoran
Group 1a	0.363	0.207	1.135	0.143
Group 1b	0.530	0.317	2.195	0.189
Group 3	N/A	N/A	3.546	0.638

Table 6.4-3
Corcoran Crossover Structure Track Serviceability Results (2)

Group	Rotation about Vertical Axis (rad)		Rotation about Transverse Axis (rad)	
	Limit	Corcoran	Limit	Corcoran
Group 1a	0.0007	0.00010	0.0012	0.0007
Group 1b	0.0010	0.00010	0.0017	0.0007
Group 3	0.0021	0.00030	0.0026	0.0020

Table 6.4-4
Corcoran Crossover Structure Track Serviceability Results (3)

Group	Deck Twist (rad)	
	Limit	Corcoran
Group 1a	0.0011	0.00081
Group 1b	0.0011	0.00052
Group 2	0.003	0.00081
Group 3	0.003	0.0019

Table 6.4-5
Corcoran Crossover Structure Track Structure Interaction Results (1)

Group	Relative Longitudinal Displacement	
	Limit	Corcoran
Group 4	1.582	0.614
Group 5	2.622	0.733

Table 6.4-6
Corcoran Crossover Structure Track Structure Interaction Results (2)

Group	Relative Vertical Displacement	
	Limit	Corcoran
Group 4	0.25	0.151
Group 5	0.50	0.262

Table 6.4-7
Corcoran Crossover Structure Track Structure Interaction Results (3)

Group	Relative Transverse Displacement	
	Limit	Corcoran
Group 4	0.08	0.003
Group 5	0.16	0.003

Table 6.4-8
Corcoran Crossover Structure Track Structure Interaction Results (4)

Group	Permissible Axial Rail Stress	
	Limit	Corcoran
Group 4	±14	4.9.0 to -4.7
Group 5	±23	13.4 to -15.8

6.4.4 Force Results

The key components of the crossover structure have been checked for structural adequacy to assess the validity of the section sizing. In addition to the crossover structure, sections of the typical viaduct which interface with the crossover were also checked. This included the box girder spans immediately adjacent to the crossover and the columns supporting these spans.

The force checks were comprised of the RC design of the columns and pile caps, feasibility of the post-tensioned cross beams at the specified sizes; and the RC design of the piled foundations.

Table 6.4-9
Column Strength Check –Load Case, Axial, and Flexural

	Strength 1	Strength 5
Axial Demand (k)	2,816	1,117
Shear SRSS (k)	1,580	1,154
Moment M3 Demand (k-in)	26,110	66,414
Moment M2 Demand (k-in)	86,317	60,430
Moment SRSS Demand/Capacity Ratio	0.616	0.58

Table 6.4-10
Pile Strength Check – Governing Axial/Moment Load Interaction

	Governing Pile Strength 5
Governing Axial Demand (k)	989
Flexural Demand (k-in)	311211
Demand/Capacity Ratio	0.960

6.4.4.1 Dowel Forces

The dowel elements have been modeled as nominal 12-inch-diameter steel pins, with 2 dowels placed at each joint on the crossover. The intermediate joints on the crossover are much broader than those between the crossover and typical viaduct, which would permit a greater number of dowel elements to be installed; reducing dowel stresses and diameters. The merits of a greater number of dowels can be evaluated in further design developments. For consistency, a two dowel configuration has been maintained in all joint locations in the structure.

The forces in the dowels have been determined and compared with the capacity of the 12-inch-diameter steel sections. In all load cases and configurations, the strength of the dowels has been found to be satisfactory. See Table 6.4-11 for a summary of the results.

Table 6.4-11
Corcoran Crossover Structure Dowel Capacity Results

Load Case	Shear Force, V3		Bending Moment, M2	
	Capacity, V_r	Corcoran	Capacity, M_r	Corcoran
Strength 1	2624	1595	6786	6535
Strength 5	2624	473	6786	3779

6.4.4.2 End-Span Check

The adoption of dowel connections at the joint between the crossover structure and the standard viaduct results in forces being transferred from the crossover into the immediately adjacent viaduct spans (end spans). These spans have therefore been checked for structural adequacy as part of the overall viaduct assessment.

The main variation between the end-spans and the standard spans is the torsional force that is induced in the box section due to the transverse load transfer from the dowels. The effects of the connection on the moments about the minor axis of the box section were also evaluated.

The check was conducted as a comparison between the shear stresses observed in the box section webs from the standard 120-foot-span sections, and the stresses in the Corcoran viaduct's end-span section. The shear stresses were derived from the applied shear and torsion forces, determined using the Corcoran SAP models. The stresses in a 120-foot standard span were taken from the boundary spans in the Corcoran crossover models.

The comparison shows that the forces transferred from the crossover increase the shear stresses in the webs of the end-span box girder by 20%. This is a manageable increase that can be accounted for by a modification of the box girder shear reinforcement. In the top and bottom flanges, the maximum shear stress increases by 65%, but it should be noted that the stresses in the flanges of the typical sections are initially small, and so stresses in the end-span sections are similarly small.

The design moments about the minor axis of the standard box girder are shown to decrease in the end-span section. This is attributed to the reduced fixity of the end-span provided by the column immediately adjacent to the crossover structure. As the bearings for only the one span are located over the centroid of the end column, in comparison with a typical intermediate column that is loaded eccentrically and by two spans, there is less resistance to the twisting of the column. When this is considered in terms of the transverse plane of bending, the end-column represents a pinned support in comparison to the rigid typical column supports. The maximum transverse moments in the column are therefore observed to be 25% less than those of the typical viaduct sections.

6.4.4.3 Thermal Load Effects

In developing the structure model it was initially thought that a 300-foot spacing of joints between panels of the structure slab would be satisfactory as this was close to the maximum thermal length requirements of TM2.10.10. Initial test analysis runs showed that, contrary to the established design philosophy of the CSDC, seismic induced loading would not be the primary driver of the design for these structures. These analyses showed that the design was primarily governed by both the frequency requirements of TM 2.10.10, and thermal loads when applied in the Strength 1 load combination.

Thermal loading was particularly dominant due to the rigid restraints provided by the columns to the superstructure, resulting in large forces being transferred from the column cap into the columns. This restraint also had the effect of constraining the thermal expansion of the superstructure at the ends of each respective thermal unit, resulting in downward hogging during thermal expansion and uplift during contraction.

To moderate these forces the model of the structure was revised to incorporate more frequent joints, typically at 150 to 180-foot spacing. This had the effect of substantially reducing the thermal effects. It is still possible that thermal loads may be the governing design case though it is more likely that the seismic case will govern the design. Where thermal forces govern the design it may be prudent for the detailed designers to consider refining the joint spacing to further reduce the thermal forces.

The dowel connections between panels were also susceptible to high loads in the Strength 1 combination. The constraints provided by the dowels have the potential to restrict the natural thermal movement of the structure both transversely and vertically, due to the uplift/hogging effects. For this reason the dowels have been articulated to allow vertical displacement and rotation about the transverse axis in an effort to reduce the forces imposed in the elements while retaining the benefit of lateral restraint from having the dowels. The increase in numbers of joints as described above also led to a substantial reduction in dowel forces. Relative displacements and rotations between adjacent thermal units were found to be within limits with this configuration.

6.4.5 Seismic Results

6.4.5.1 Columns

The displacement capacity and displacement ductility demand of the columns were assessed in accordance with the CSDC. Two percent reinforcement ratio is provided for the 6-foot elevated slab columns. The reinforcing steel detail is shown in Figure 6.4-3. The assessment was completed using a combination of global and local SAP models and running pushover analyses in both the X and Y directions.

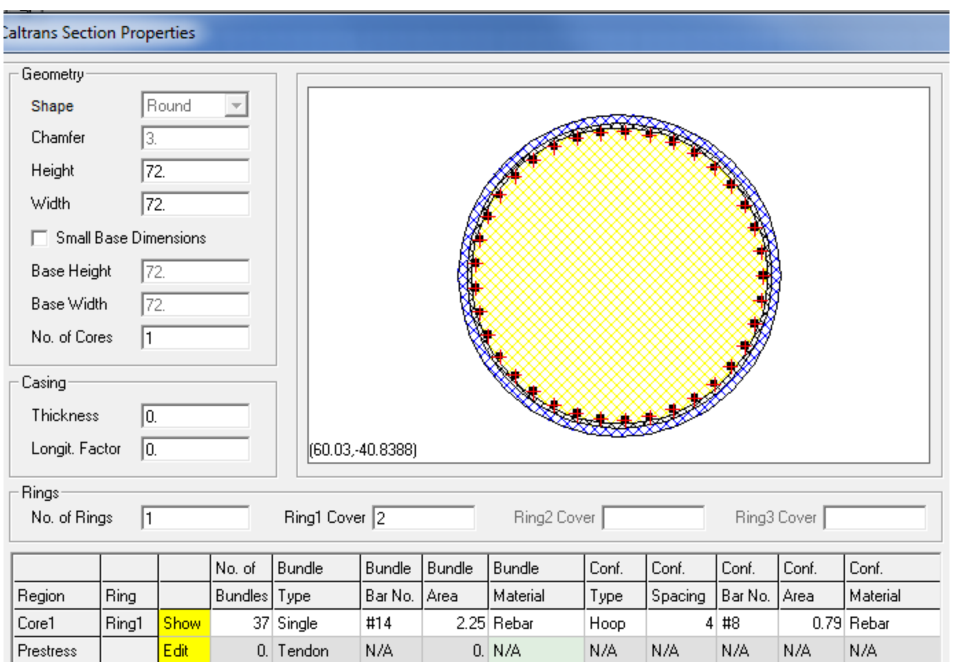

Figure 6.4-3
Section Properties – Corcoran Crossover Structure 6-foot Elevated Slab Column

Due to the number of columns, pushover analysis proved to be difficult, as the software struggled to record the exact yield and collapse states for each and every column. Columns are seen to yield at different pushover steps. In displacement ductility demand check, the pushover step that presents the first column yield is taken as the reference and only those columns that yield during this step are reviewed for displacement ductility. In the case of the ductility capacity check this was not an issue as only a local model of a single column was required.

It has been demonstrated from the analysis and calculations that the Corcoran Crossover Structure is structurally viable and meets the requirements of the CSDC. See Tables 6.4-9 to 6.4-12 for a summary of the results.

Table 6.4-12
Seismic Displacement Check – Displacement Demand Ductility Check

	Displacement Ductility Upper Limit	MCE Displacement ΔD (in)	Yield Displacement ΔY (in)	Displacement Ductility, μD
Longitudinal	5	2.2489	2.2613	0.995
Transverse	5	2.0911	2.3054	0.907

Table 6.4-13
Seismic Displacement Check – Capacity Ductility Check

	Capacity Ductility Lower Limit	Yield Column Displacement ΔYCOL (in)	Collapse Column Displacement ΔcCOL (in)	Capacity Ductility, μC
Longitudinal	3	0.274	1.438	5.25
Transverse	3	0.272	1.448	5.33

Table 6.4-14
Seismic Displacement Check – Displacement Demand/Capacity Ratio Check

	MCE Displacement Demand ΔD (in)	Displacement Capacity Δc (in)	Demand/Capacity
Longitudinal	2.2489	3.6993	0.61
Transverse	2.0911	3.7534	0.56

Table 6.4-15
Column Strength Check

	Envelope
Plastic Moment, Mp (k-in)	185,519
♦ **Overstrength Moment, Mo (k-in)**	222,623
♦ **Overstrength Shear Demand, V (kips)**	1400
Shear Capacity ØVn (kips)	2280

6.4.5.2 Foundations

The demand forces to be used for the foundation design are specified in the CSDC and include the service level moments, shears, axial loads and the moment demand induced by the column plastic hinging mechanism. The moment derived from the plastic hinging mechanism is taken as the plastic moment of the column, multiplied by an overstrength factor of 1.2 to give the overstrength moment. The adoption of the overstrength factor is based upon the seismic design philosophy whereby the columns will always yield at the MCE event to protect the pile. The factor is required to account for risk that the column may develop a greater plastic moment capacity than the idealized values used in the design. An axial-moment diagram from the section designer module in CSiBridge was used to check the reinforcement design of the pile. A minimal reinforcement ratio of 1% and #8 at 4-inch tie spacing was used for 9-ft diameter pile. Pile shear and moment demand derived from column overstrength moment are checked against capacity. Pile shear and moment demand derived from column overstrength moment are checked against capacity.

See Table 6.4-10 for a summary of the results.

Table 6.4-16
Pile Strength Check – Column Overstrength Demand per Pile

	Max Axial	**Min Axial**
Maximum Axial Demand	1,242	-1,180
Overstrength Design Shear Demand (k)	1,769	1,320
Shear Capacity (k)	4719	4719
Shear Demand/Capacity	0.37	0.28
Design Moment Demand per pile (k-in)	194,563	145,244
Moment Capacity per pile (k-in)	315,739	237,843
Moment Demand/Capacity	0.616	0.611

6.5 Limits of Standard Bridge Design and Special Bridge Design

The boundary spans as mentioned in Section 6.4.4.2 have the standard length and cross section and considered as standard structures. Therefore, the standard bridge design is suitable for use

on spans 1 to 13 and from spans 15 to 32. The crossover structure itself occupies the section of viaduct between bent 14 and bent 15 and is the subject of this analysis.

6.6 Construction Methods Assessment

The assumed method and sequence of construction for the crossover structure is to construct the CIDH shafts alongside the BNSF right-of-way line. These piles will be extended as columns in a second stage concrete pour. Subsequently it is assumed that the lower part of the column cap beam will be formed and cast on falsework to provide a temporary seat onto which the precast beams can be placed.

It is assumed that beams will be lifted from the east side of the structure straight from the delivery truck using a mobile crane. It is expected that as beam placement is a relatively quick operation, this can be done between trains. The BNSF should be consulted to confirm the acceptability of this approach. Some beams adjacent to expansion joints may require additional concrete for the joints to be cast onto them as a second stage pour prior to erection.

Each beam will have a lift weight of approximately 60 to 70 tons and the erection lift radius is likely to be approximately 100 to 130 feet.

Once a section of beams between expansion joints is placed, the deck slab in that area can be cast to produce the final slab structure. Stay in place forms soffit forms will be required between beams per BNSF guidelines. The deck pour is also assumed to include the upper half of the column capping beam which allows the beams and deck to act monolithically with the column cap and columns.

The constraints specific to the crossover structure suggest that a particular method of erection is most likely to be used by contractors. This does not rule out other methods of construction. It is likely that contractors will prefer to use methods that they have used successfully in the past. The assessment described here represents a subset of methods that could be used.

6.7 Temporary Construction Loadings Considered

No specific loadings have been considered for the temporary stages described.

6.8 Temporary Construction Easements

A general temporary construction easement of 100 feet width has been identified for the full length of the crossover slab on the side remote from SR 43. It is expected that this will be sufficient to accommodate the access and crane requirements for beam placement over the BNSF; however, placing the beams over the SR 43 traveled way will require traffic management and control to be agreed with Caltrans.

Provision has been made for temporary construction easements of 15 feet width to both sides of the proposed HSR right-of-way boundary where the standard viaduct is used.

6.9 Traffic or Pedestrian Diversion and Control

The construction of the standard viaduct is expected to be supplied from along the HSR route. At the crossover structure there is access to the local roadway system from SR 43, which runs parallel for its entire length. SR 43 is likely to be the primary access route to the locality of the structure for supplying beams and construction materials to the worksite. Temporary haul roads may be required to access the remote side of the structure. As this is a rural area it is anticipated that only localized traffic control measures will be required at the site entrances at peak work times.

6.10 Drainage Concept

The track drainage for the Corcoran Viaduct will be carried from deck level through to a permanent drainpipe fitted within the void of the concrete deck girders. This pipe will be connected to downpipes cast into the columns. The downpipes will outfall near ground level to the surface drainage system.

For the crossover structure, provision will be made for collecting water at track level. This will be conveyed to the ends of each thermal unit of the deck slab via a longitudinal carrier pipe that will be located within the track bed. At the ends of the thermal unit the carrier pipes will direct flow towards the edge beams and discharge through the expansion joints to the nearest available downpipe.

6.11 Emergency Access Provision

Provision for emergency access will be made in accordance with TM 2.8.1 Safety and Security Design Requirements R0 (March 12th, 2012), TM 2.3.3 HST Aerial Structure R0 (June 2nd, 2009), NFPA130 and NFPA101. Emergency access points are required at maximum 2,500-foot intervals along aerial structures with access stairs to be located every 2.5 miles. It is also a requirement that access to the trackside is provided at each systems site which are also at approximately 2.5 mile intervals. Therefore, access stairs have been provided at each systems site and emergency ladder access turnarounds are provided at 2,500-foot nominal centers between systems sites. The Corcoran Viaduct is not of sufficient length to require emergency access stairs.

Table 6.11-1
Escape Stair Locations

STA	Locale	Egress features
2996+40	Adjacent to Popular Ave / SR 43	Access stairs near to systems site

6.12 Inspection, Service, and Maintenance Access

The standard viaduct will be a simple concrete section which can be inspected from both inside and outside.

The crossover beams are envisioned to be placed immediately adjacent to one another and cast into the edge beam at their ends. These are unlikely to be inspectable as they will also be over the BNSF corridor and they should be designed with this in mind. There will need to be an agreement between the Authority and the BNSF to provide access for future inspection and maintenance during non-revenue hours.

Externally, the crossover structure will be inspectable with the use of hydraulic access platforms either from grade or above.

6.13 Utilities Affected and Disposition

Refer to composite utility plans.

6.14 Noise Mitigation and Acoustic Treatment

No specific features have been included in the structure to mitigate the noise generated by the passage of trains. The project environmental impact statement (EIS) will define what measures are required to mitigate noise impacts.

However, this structure is considered sufficiently robust to accommodate the addition of, for example, noise protection fencing, should this be required for mitigation of impact.

6.15 Compliance with System-Wide Bridge Aesthetics Features

TM 200.06, "Aesthetic Guidelines for Non-Station Structures" provides guidance on the appearance targets for the CHSTP. The scheme detailed on the PE4P drawings and analyzed represents the functional baseline case on which the DB contractors are encouraged to improve in discussion with the Authority.

6.16 Geotechnical Parameters Used for Design

The geotechnical parameters are described in the Geotechnical Design Memorandum attached at Appendix A.

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Section 7.0

Other Structures

7.0 Other Structures

Paragraph 1.3 describes the method of classification of structures for design. Under this classification the standard structure is the 120-foot-span viaduct girder. The bulk of this report has been concerned with Complex and Nonstandard parts of the major viaducts. However, the simpler structures required for hydraulic crossings, wildlife crossings and retention of embankments in constrained locations fall under the description of nonstandard structures and so are pertinent to this report. As the majority of these structures also support the HSR directly they are also classified as Primary Structures.

7.1 Box Culverts

The locations of hydraulic crossings have been identified on the 15% Record Set Drawings as an indicative centerline. No additional site-specific details have been developed for these structures for the PE4P design as the requirements for each crossing are subject to agreement with the relevant Irrigation or Flood Control Districts that have jurisdiction in the locale.

General details of typical culvert structures have been developed to inform the PE4P bidder. These details cover typical forms of culvert structure that may be required and include 1 cell, 2 cell and 3 cell structures.

The detailed designers should note that if multicell structures are proposed, the risk of the dividing walls collecting debris during flood conditions will be a concern of local jurisdictions. The structure is likely to require additional measures to prevent such debris from obstructing flow, such as inclined "cut-water" walls that encourage debris to be pushed up above the flow during flood.

Local jurisdictions are likely to require the ability to dam off the structure to permit maintenance and inspection access to the internal surfaces. The typical details indicate grooves for the insertion of stop beams, but the precise details and locations of these features should be agreed during detailed design.

Preliminary section sizing has been carried out on the basis of a range of cells (1 to 3 cells), spans (10 feet and 15 feet), heights of openings (5 feet or 10 feet) and height of embankment above the top of the structure from 6 feet to 30 feet. These dimensions are summarized on the typical details drawing. The minimum cover to the top of the structure required by the design criteria is 6 feet.

No specific seismic analysis has been conducted as the peak ground acceleration in the CP2-3 area is less than 0.35g. It is therefore assumed that the structure sections will be designed for "at-rest" lateral earth pressures.

Typical cross section details are shown in Figures 7.1-1 to 3 below.

7.1.1 Construction Details

The design concept makes no specific assumption regarding the construction of culvert structures and it is likely that a variety of methods will be used throughout CP2-3. Where the construction site is in open country with the ability to use temporary diversions over long periods, the structures may be constructed in situ. Where the site is constrained either in extent or time available for construction, then precast culvert segments are likely to be favored. The 15% footprint makes allowance for temporary construction diversions at the location of all major hydraulic crossings.

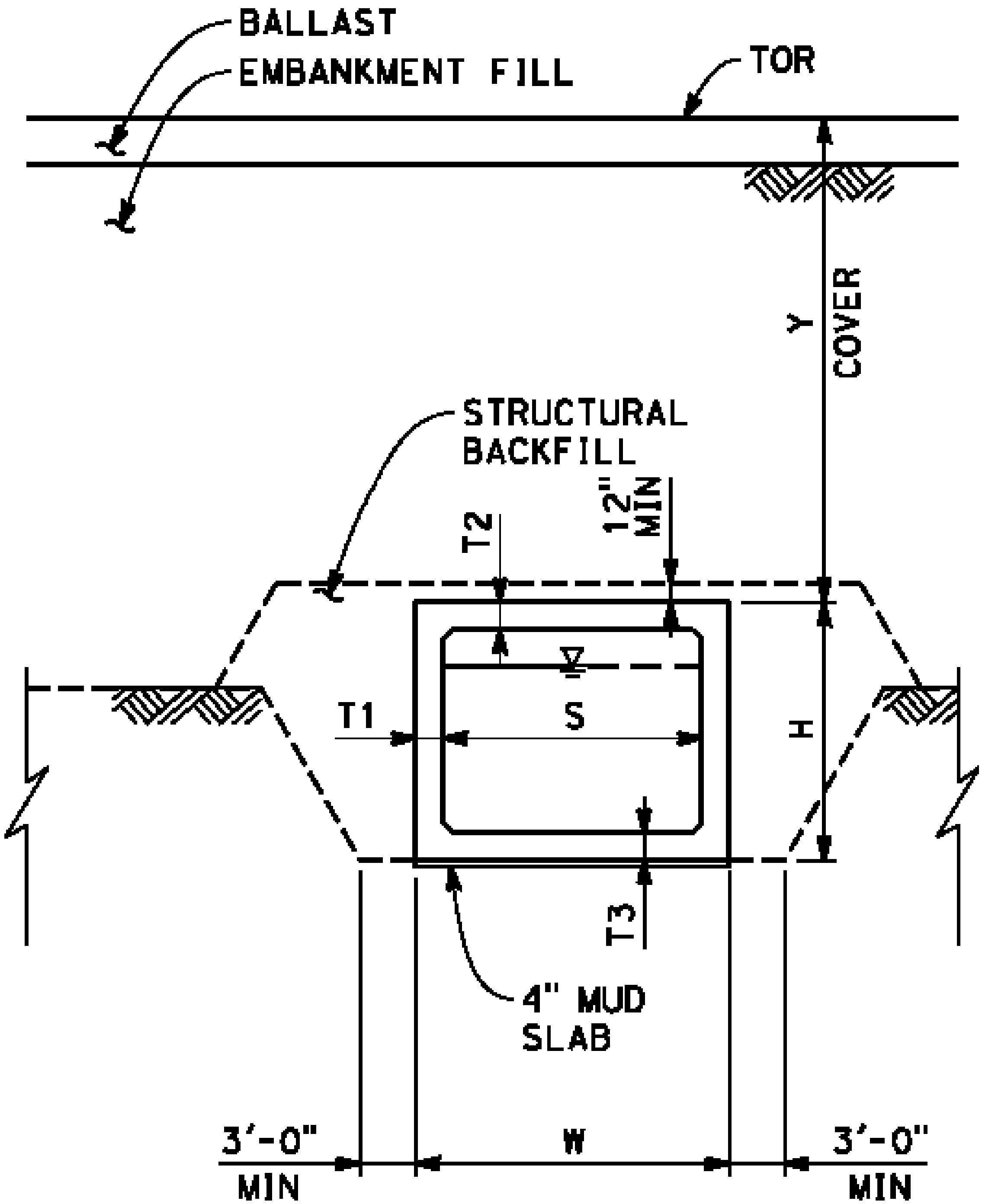

Figure 7.1-1
Typical Section – Single Cell Box Culvert

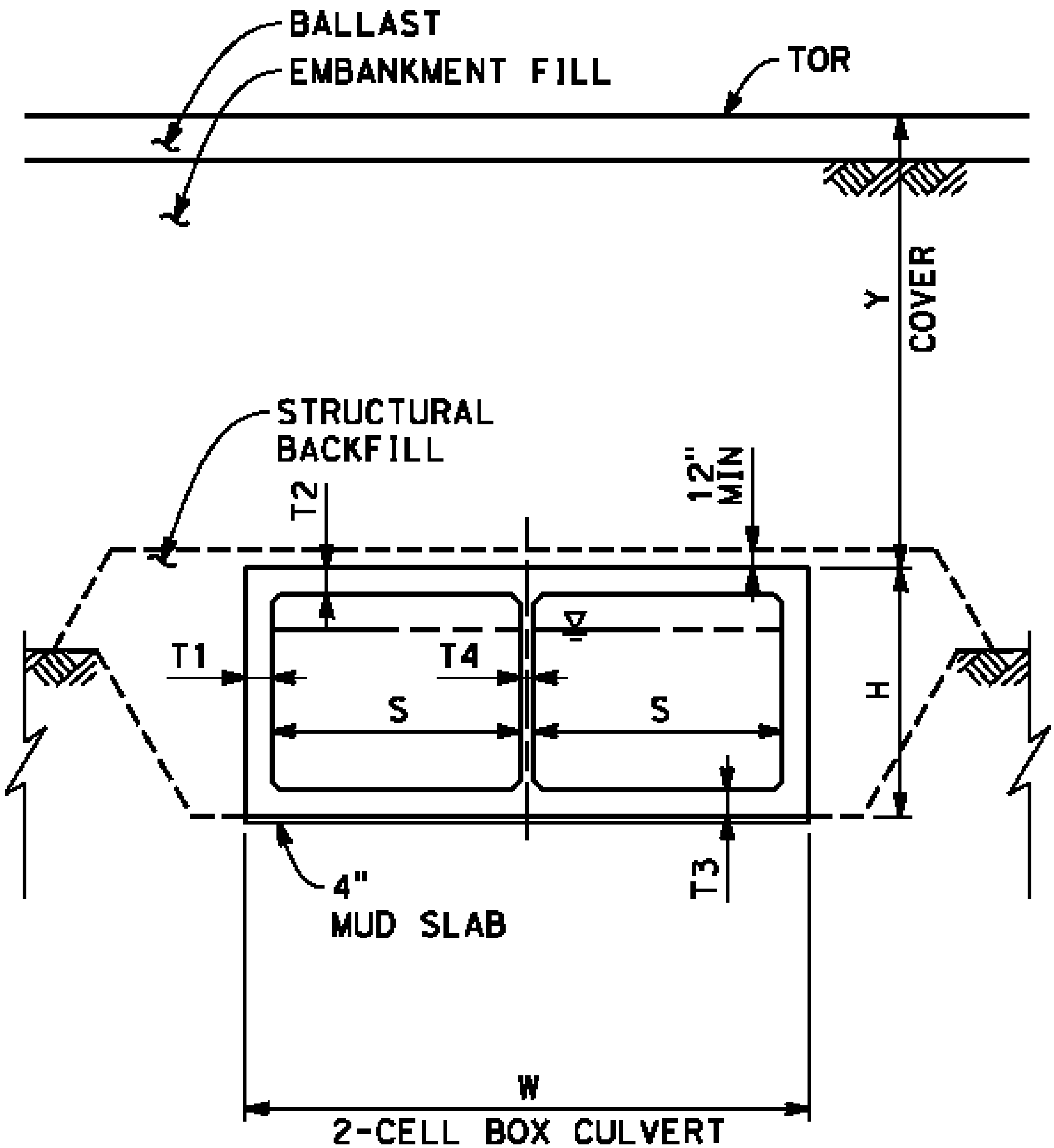

Figure 7.1-2
Typical Section – Two Cell Box Culvert

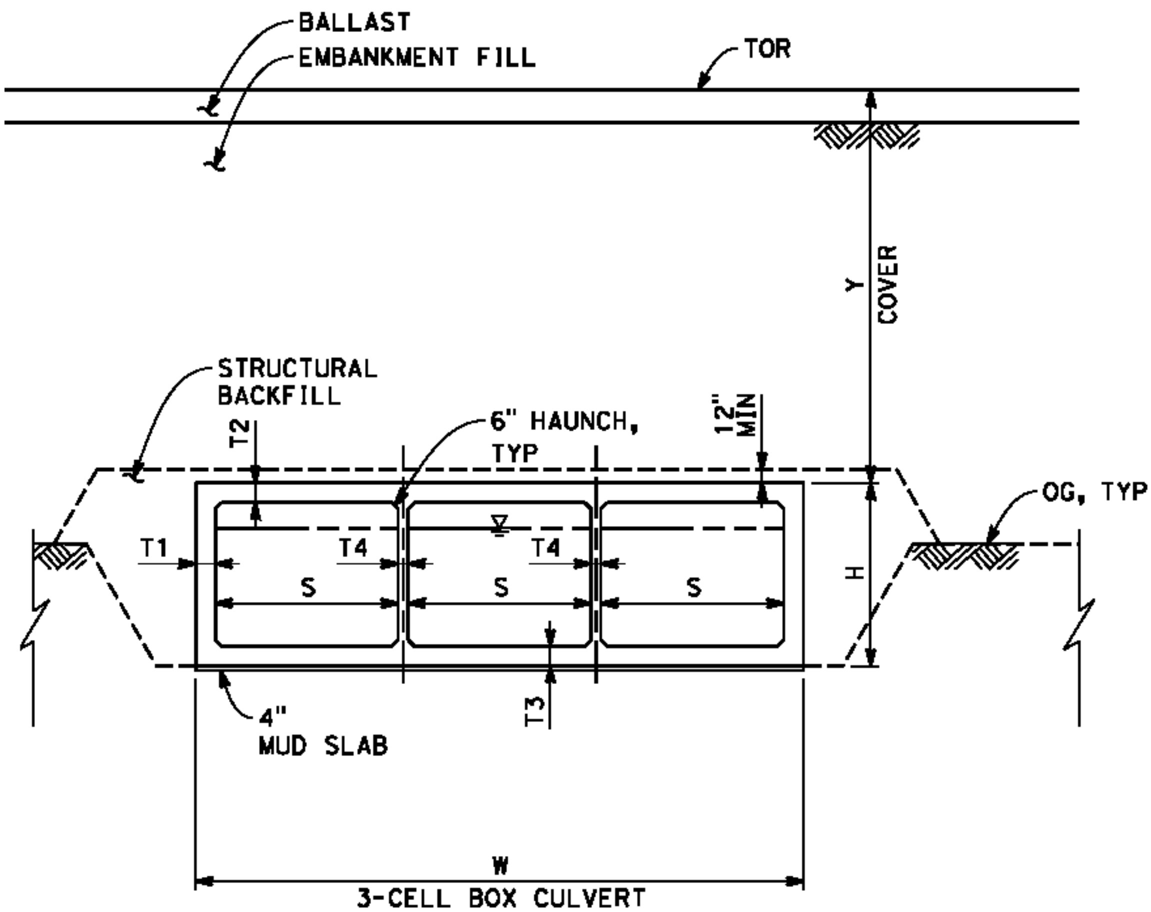

Figure 7.1-3
Typical Section – Three Cell Box Culvert

7.2 Wildlife Crossings

Details of the requirements for wildlife crossings are presented in the environmental report. In general these structures are wide to be nonthreatening to wildlife and have a low internal height.

Structurally these structures are similar to box culverts and the provisional details indicated above should be sufficient to form the basis of a detailed design.

7.3 Retaining Walls for Retained Embankments

Where the HSR rises above adjacent grade there is typically a transition from embankment to viaduct which includes a length of retained embankment. This transition zone has been adopted in the preliminary design at the direction of the Authority's representatives to limit the footprint requirements of the project.

The typical form of these retained embankments is assumed to be a Mechanically Stabilized Earth (MSE) system. There are many commercial systems that are capable of being designed to provide this function so there has been no attempt to specify detailed requirements for these structures that may result in exclusion of some suitable products.

Due to the lack of available GI information, no assessment has been made regarding whether the subgrade soils would need to be treated either by ground improvement or piled raft foundations in order to support these structures. The 15% record set drawings indicate that piles may be required.

Typically the retained embankments begin at an embankment height of 15 feet and end at a viaduct abutment with a retained height of approximately 30 to 35 feet. This means that as the retained HSR corridor width is approximately 60 feet wide the height to width ratio of these embankments is generally much less than 0.5 and so global stability is not considered to be critical for design.

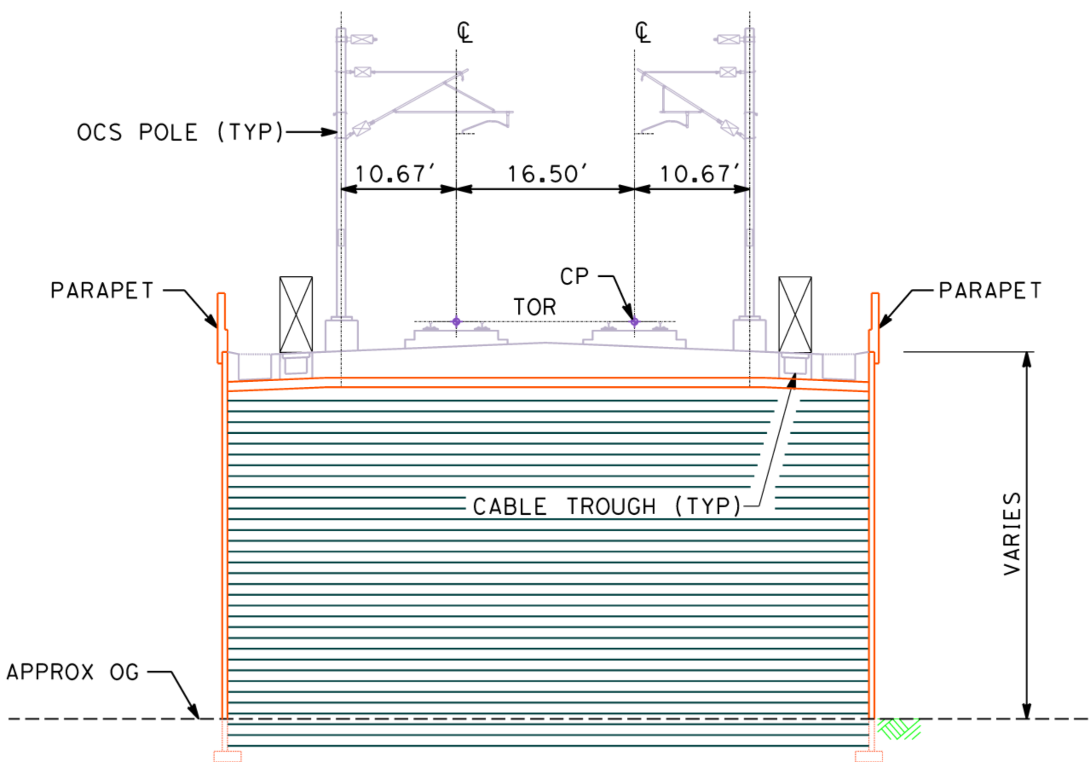

Figure 7.3-1
Typical Section – Retained Embankment

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Section 8.0

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8.0 References

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Appendix A

Geotechnical Design Report

APPENDIX A Geotechnical Design Report for Nonstandard and Complex Structures

Attached is the geotechnical design memorandum, which is based on information gathered from historic Caltrans borehole data from the vicinity of the HSR route. The interpretation if the information gathered has been summarized to provide conservative lower-bound properties for the analysis of the structures.

A project-specific GI is underway that will supplement the information in this memorandum. It is expected that this GI will enable the soil parameters to be modified to be less conservative than has been used in the analyses. It is anticipated that this will enable the structures designs to be made more economical.

APPENDIX A

Fresno to Bakersfield
Package 2-3

Geotechnical Design Report
for
Nonstandard and Complex
Structures

Prepared by:

URS/HMM/Arup Joint Venture

June 2014

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CALIFORNIA High-Speed Rail Authority

List of Abbreviations and Acronyms

AASHTO	American Association of State Highway and Transportation Officials
ASD	Allowable Strength Design
bgs	below ground surface
Caltrans	California Department of Transportation
CBC	California Building Code
CBDS	California Bridge Design Specifications
CDMG	California Division of Mines and Geology
CGS	California Geological Survey
CIDH	cast-in-drilled-hole
HST	California High-Speed Train Project
deg	degrees
DWR	California Department of Water Resources
EL	elevation
EMT	California High-Speed Train Engineering Management Team
ESS	Excavation Support System
FOS	factor of safety
g	gravity
GPS	Global Positioning System
GWL	groundwater level
HMM	Hatch Mott MacDonald
HST	High-Speed Train
in	inches
JV	HMM/URS/ARUP Joint Venture
K	USDA Soil Erodibility Factor
ksf	Kips per Square Foot
LL	lower limit
LRFD	Load and Resistance Factor Design
MCL	Maximum Considered Earthquake
mi	miles
mm	millimeters
M_W	Moment Magnitude
$(N_1)_{60}$	Standard Penetration N-Values Corrected for Hammer Energy, Overburden Pressure, and Field Procedures
N_{60}	Standard Penetration N-Values Corrected for Hammer Energy
NAD27	1927 North American Datum
NAVD88	1988 North American Vertical Datum
NA	not applicable
NCL	Non-Collapse Performance Level
NRCS	Natural Resources Conservation Service
OBE	Operating Basis Earthquake
pcf	pounds per cubic foot

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pci	pounds per cubic inch
PGA	peak ground acceleration
PMT	California High-Speed Train Project Management Team
SJV	San Joaquin Valley
SPT	Standard Penetration Test
SPT N	Standard Penetration Test Blow Count
Sta	Station
T	Period
TM	Technical Memorandum
UBC	Uniform Building Code
UL	upper limit
USGS	United States Geological Survey
USDA	United States Department of Agriculture
$(V_S)_{30}$	Average Shear Wave Velocity in the upper 30 meters of soil
yr	year

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Section 1.0

Scope

1.0 Scope

1.1 Appendix Purpose

This appendix is presented as a geotechnical design report to address non-standard and complex structures in Construction Package 2-3 (CP2-3). CP2-3 comprises a subsection of the alignment from E American Avenue (STA 557+00) to south of the Fresno metropolitan area to 1 mile north of the border of Tulare County and Kern County (STA 4435+50).

This appendix presents geotechnical design calculations and recommendations calculated under a scope for Preliminary Engineer for Procurement (PE4P). The design presented in the main text of this report reflects the design of these structures as undertaken in 2011 and 2012. The design was based on geotechnical data available at that time; composed entirely of historical geotechnical information undertaken by others (primarily Caltrans sources).

Although execution and reporting of recent site-specific geotechnical investigations has become available during the development of this Appendix, this data have not been reviewed or considered in the design. The quality and coverage of the historical data used in the design is poor, limited to historical Caltrans boring logs often more than a mile from the proposed alignment. Thus there is potential that the recently collected data would lead to design changes. Re-design based on the new geotechnical information, and any resulting changes to the reference design, are outside the scope of services; however a conclusion is drawn in this appendix of the applicability of the design presented herein considering the data obtained from field geotechnical investigations in late 2013 in Fresno and Tulare counties.

1.2 Relevant Structures

A list of the non-standard and complex structures of Construction Package 2-3 is provided in Table 1.2-1 below. Refer to the Sierra Subdivision Construction Package 2-3 Non-Standard and Complex Structure Report for further details.

Table 1.2-1
Relevant CP2-3 Structures

No.	Purpose	Structural Type	Structure Class	Location (Beg. Station)	Length
3	Conejo Crossover Structure	Crossover Beam/Slab Structure	Non-Standard	1120+30	1,429
8	Kings River Viaduct	Truss Span Single span (1 bent)	Complex	1464+80	217
10	Cole Slough Bridge	Steel Truss Single span (1 bent)	Complex	1485+60	357
12	Dutch John Cut	Steel Truss Two spans (1 bent)	Complex	1518+20	714
14	Kings River Bridge	Steel Truss Two spans (1 bent)	Complex	1581+17	644
16	Levee Road Bridge	Steel Truss Single span (1 bent)	Complex	1593+64	283.5

Table 1.2-1
Relevant CP2-3 Structures

No.	Purpose	Structural Type	Structure Class	Location (Beg. Station)	Length
19	Hanford Viaduct (including Kings/Tulare Regional Station)	PC Girder, precast standard spans, precast nonstandard spans	Non-Standard & Standard	1903+57	10,480
21	Kaweah SR 43 Crossing	Steel Truss Two spans (1 bent)	Complex	2240+32	574
24	Cross Creek Viaduct	Steel Truss Single span (no bents)	Complex	2479+22	322
26	Cross Creek Viaduct	Crossover Beam/ Slab Structure	Non-Standard	2530+00	525
29	Whitley Ave/SR137	Steel Half Through Girder Single span (no bents)	Non Standard	2812+76	90
32	Corcoran Crossover Structure (part of SR 43/BNSF Viaduct)	Crossover Beam/ Slab Structure	Non-Standard	3005+00	2,426

Section 2.0

Physiography and Geologic Setting

2.0 Physiography and Geologic Setting

2.1 Physiography

The topographic provinces included in this report are based on the USACE topographic map (1962) and the physical model GIS layer. These reference sources cover the Fresno-to-Bakersfield portion of the alignment in its entirety, but are out of date. Project-specific surveying data and light detection and ranging data, to be completed in the future, will be used to more fully describe the physiography and topography of this segment of the alignment.

As described by the Fresno to Bakersfield Geologic and Seismic Hazards Report (GSHR 2013), the HST segment between Fresno and Bakersfield is located fully within the San Joaquin Valley (SJV) at an elevation between about 200 and 400 feet above sea level (ASL) and passes through gently undulating low relief terrain with shallow natural slopes through the urban areas of Wasco, Shafter and Bakersfield. From the alignment's intersection with Highway 99 to the terminus of the study area east of Edison the topography gently rises from about elevation (EL) 400 to 680 feet ASL as it flanks the southern foothills of the Tehachapi Mountains. The general physiography and topography of the SJV within the study area is shown on Figure 2.1-1. Superimposed upon this large-scale, relatively flat topography is a localized topography of river systems caused by recent incisions. This localized topography comprises short steep river/stream banks with channels at lower elevations relative to the surrounding areas. These channel bottoms range between wide, relatively flat-bottomed (with occasional rounded natural levees) and narrow gully-type valleys, depending on their age and the amount of flow.

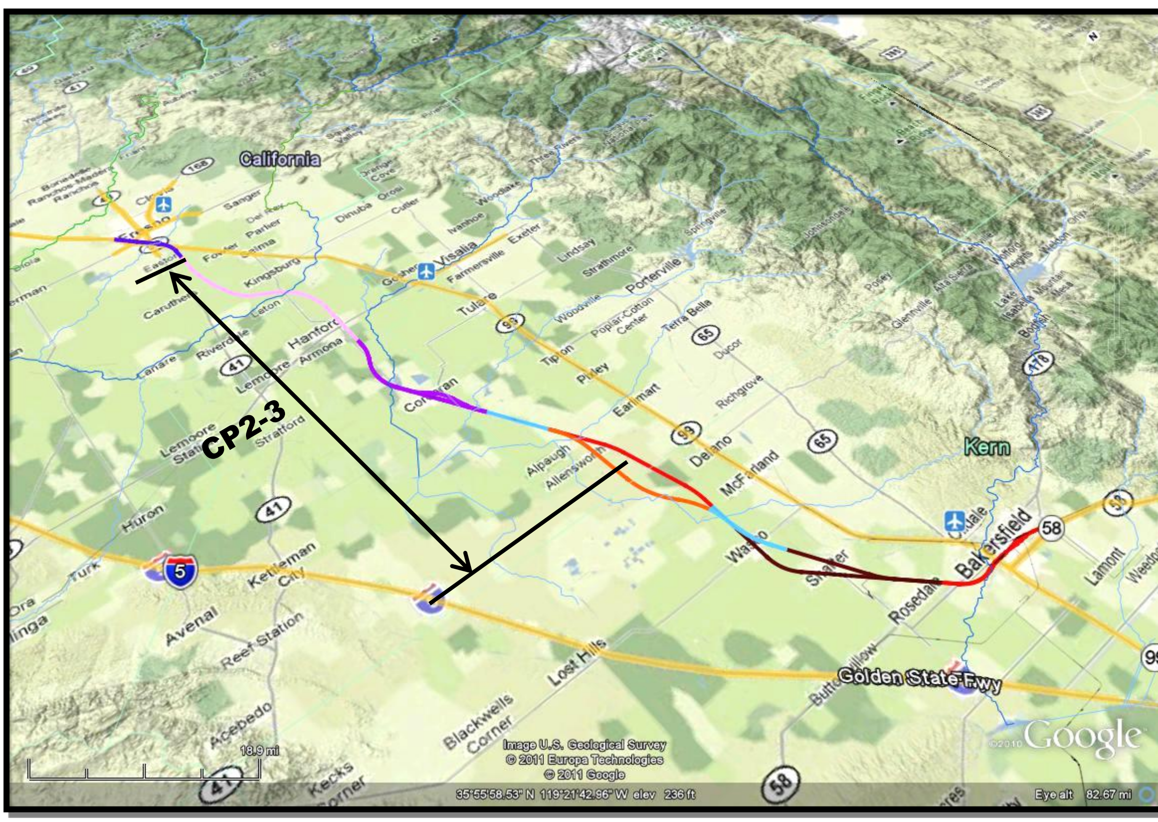

Figure 2.1-1
General Study Area Physiography and Topography (© 2011 Google Inc., 2011)

The topography along the Construction Package 2-3 corridor is generally flat and varies between elevation (EL) 295 and 205 feet relative to the North American Vertical Datum of 1988 (NAVD88). Localized variations on the ground surface elevation occur at existing road embankments, detention basins, and other man-made features such as irrigation canals and road and rail crossings.

2.2 Geologic Setting

The SJV comprises the southern part of the approximately 400-mile-long Great Valley geomorphic province. The Great Valley geomorphic province is an asymmetric synclinal trough that is filled with sediments up to 30,000 feet thick. Infilling with sediments has occurred since the Jurassic period (>145 million years), providing a large, flat-lying alluvial plain setting in which the FB alignment corridor will be constructed. Bordering the Great Valley are mountain ranges, principally the Sierra Nevada ranges that represent the Sierra Nevada geomorphic province to the east, and the Temblor and Diablo ranges associated with the Coast Ranges geomorphic province to the west (Figure 2.2-1). The Tehachapi Mountains and Klamath Mountains define the southern and northern limits of the Great Valley, respectively.

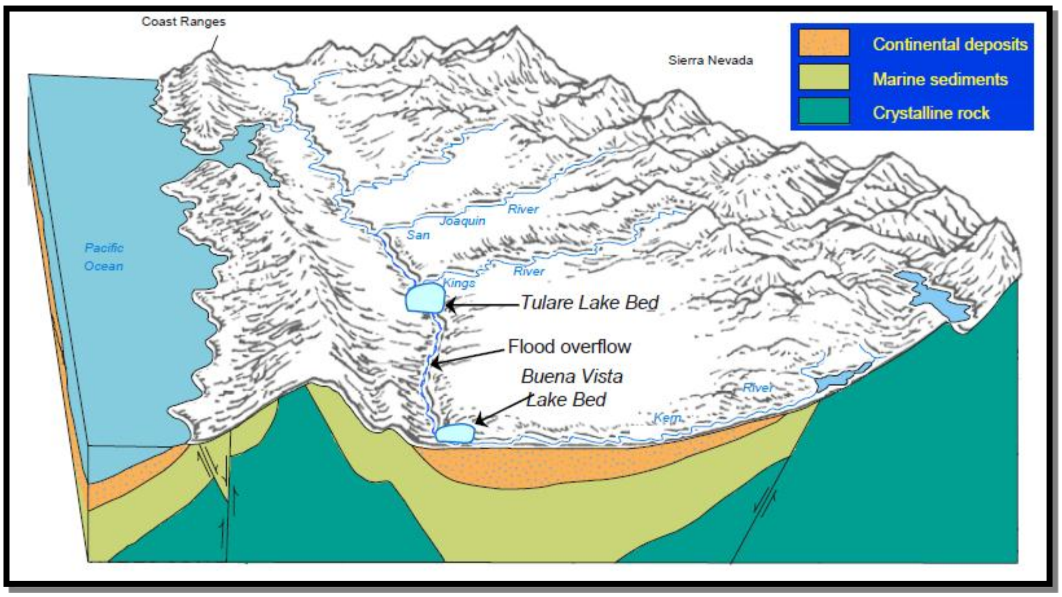

Figure 2.2-1
The Great Valley Geomorphic Province (Page, 1986)

The SJV is a large sedimentary basin, but it provides for a somewhat varied geological setting. Given the asymmetry of the synclinal trough with its axis off center to the west (Norris and Webb 1990), basin sediments are deeper on the western side of the SJV compared with the eastern side. Southwestward tilting of the trough has also contributed to greater thickness of sediments at the southern end of the SJV compared with the northern end. Bedrock geology also differs from the east to west:

To the east of the valley, the Sierra Nevada is composed primarily of pre-Tertiary granitic rocks and is separated from the valley by a foothill belt of Mesozoic and Paleozoic marine rocks and Mesozoic metavolcanic rocks along the northern one-third of the boundary. The Coast Ranges west of the valley have a core of Franciscan assemblage of late Jurassic to late Cretaceous or Paleocene age and Mesozoic ultramafic rocks. (Gronberg et al. 1998)

Such variability is testament to the tectonic environment in which the SJV is located, and the interplay that this tectonic environment has had with the formation of the SJV to the present-day.

Section 3.0

Seismic Setting

3.0 Seismic Setting

The study area is within a relatively seismically quiescent region between two areas of documented tectonic activity: the Coast Ranges-Sierran Block boundary zone to the east and the Pacific Coast Ranges boundary zone to the west.

The Coast Ranges-Sierran Block, which follows the physiographic boundary between the Coast Ranges and Great Valley geomorphic provinces, contains potentially active blind thrust faults (Unruh and Moores 1992). Based on the size of historical events and on the inferred subsection of the boundary zone, these blind thrust faults are capable of producing moderate to large earthquakes. The Pacific Coast Ranges contain many active faults that are associated with the northwest-trending San Andreas Fault System (Jennings 1994), which is the principal tectonic element of the North American/Pacific plate boundary in California.

In the SJV, seismic slip is partitioned onto subsidiary structures, such as the San Andreas, Garlock, and Coalinga Faults, which are distributed across the Great Valley geomorphic province but not in close proximity to the study area.

3.1 Faults and Seismicity

There are no known active faults crossing or within close proximity to the alignment within the study area. The San Andreas Fault, located approximately 45 miles west of the Construction Package 2-3 Alignment from the site, has the highest slip rate and is the most seismically active of any fault near the HSR alignment. While none cross the Construction Package 2-3 alignments, the White Wolf, Garlock, Kern Canyon, Edison, and Tehachapi Creek faults are deemed "capable" by HSR standards (GSHR 2013). Capable faults within the study area are presented in Table 3.1-1.

Table 3.1-1
Capable Faults within the Study Area

Fault Name	Fault Type	Slip Rate (mm/yr)	Distance and Bearing to FB HST Alignment (mi)
San Andreas	Right-Lateral Strike-Slip	20-35	47+ miles W of alignment
Clovis Fault	–	–	12 miles NE of alignment near Clovis
Kern Canyon	Normal	–	66 miles E of alignment, at Hanford
Garlock	Left-Lateral Strike-Slip	2-10	34 miles SE of alignment
White Wolf	Left-Lateral Reverse	3-8.5	13 miles SE of alignment
Edison Fault	Normal	–	Edison Fault Crossing is in Bakersfield and is discussed in Bakersfield to Palmdale alignment. Listed here for information only

3.2 Seismic Design Criteria

Procedures for defining the seismic design parameters for the HSR are defined in TM 2.10.4. The ground motion package for design will be provided by the Authority under a separate cover.

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Section 4.0

Geologic and Seismic Hazards

4.0 Geologic and Seismic Hazards

Refer to the Geologic and Seismic Hazards Report (GHSR) for detailed discussion of ground-related hazards.

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Section 4.0

Geotechnical Conditions

5.0 Geotechnical Conditions

This Appendix section presents geotechnical data and preliminary design recommendations in support of Package 2-3 structures based on historical geotechnical data along the HST study area. No site-specific geotechnical investigation was available for the preliminary design of the structures (i.e., bridges and elevated structures) in Package 2-3 of the Fresno to Bakersfield Section of the HST alignment. The JV compiled the historical data largely from (limited) Caltrans sources.

5.1 Historical Boreholes

The primary source of publicly available geotechnical data is the California Department of Transportation (Caltrans) collection of as-built construction records. Caltrans data are concentrated along SR 41, SR 43, and SR 99, from projects dating between 1953 and 1997. For each project, several boreholes were drilled, logged, and (often) plotted on a cross section. None of the Caltrans records contain laboratory test data.

Borehole records collected from Caltrans extend to a maximum depth of 122 feet below ground surface (bgs). Of the 213 historical borings included in this report, 25 borings extend greater than 70 feet bgs. The average depth is 45 feet. Five historical Caltrans sites are within 0.5 miles of the alignment, most typically where the proposed railway alignment is in close proximity to the existing SR 43 alignment. The remaining locations included in the database are between 0.5 and 5 miles from the alignment.

All relevant data from these records have been included in an appendix to the recent issue of the Geotechnical Data Report for CP2-3 (GDR). Note that the GDR includes also results of the recent project-specific geotechnical investigation, which as discussed in Section 1.0 has not been used in the development of the designs presented herein.

An indication of the coverage of historical information is provided by Figure 5.1-1 below, excerpted from the GDR. Observe the extremely large scale, and recall that the nearest data is approximately 0.0.5 miles from the alignment. The volume of data within 5 miles of the alignment has been annotated, for information.

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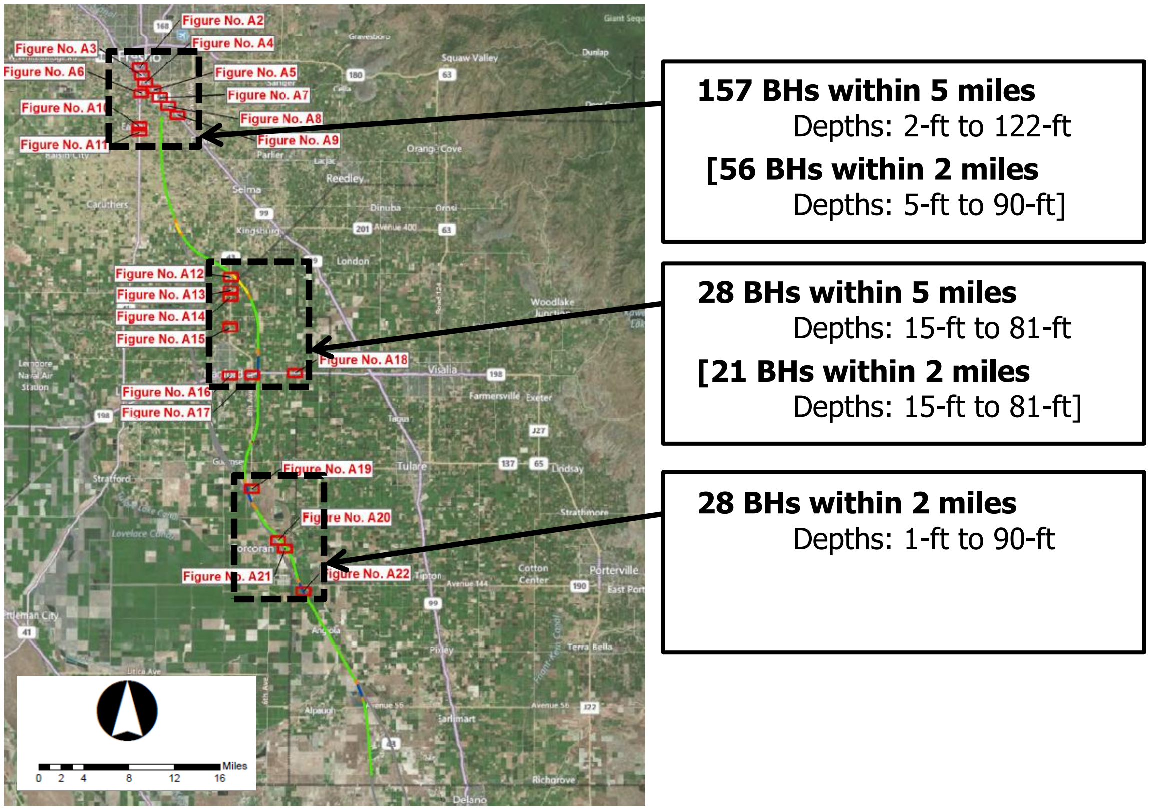

Figure 5.1-1
Coverage of Historical Geotechnical Data for CP2-3 (from 0.5 to 5.0 miles from alignment)

In total, historical data are available from 213 boreholes within 5 miles and 105 boreholes within 2 miles of the Package 2-3 alignment; a section comprising over 64 miles of track.

The available boreholes tend to be concentrated in three primary clusters, with interim gaps of from 8 to 16 miles. Furthermore, many of the borehole logs offer little detail about the soils beyond the depths of potential bearing strata for deep foundations (where relevant).

5.2 Stratigraphy

General overview of soil stratigraphy anticipated along Package 2-3 alignment has been present in GSHR Sections 3.9.2 through 3.9.7, and 3.8.2 through 3.8.7.

The GSHR divides the Fresno to Bakersfield alignment into several subsections by geomorphology and physiography. Table 5.2-1 presents a summary of the GSHR stratigraphy's, based on the very limited historical geotechnical data. The alignment subsection descriptions are not critical for this appendix, but are used to differentiate the typical stratigraphy in the table below.

Table 5.2-1
Summary of Stratigraphy Along Package 2-3 Alignment Based on Historical Data

FB-B Rural North	FB-C King River Crossing	FB-D Hanford Station	FB-E Rural Central	FB-F Tule River Crossing	FB-G Rural South (Part)
0-15 ft bgs (275-260 EL) Alternating beds of med-dense poorly graded sand and silty sand (N = 10-12)	0-55 ft (270-215 EL) Alternating beds of loose to v.dense poorly graded sand, silt, with clay (N = 6-130)	0-27 ft (265-238 EL) Alternating thin to med beds of med-dense sand (N = 14-35) and stiff silt (N = 27)	0-90 ft (210-120 EL) Alternating beds of loose to med-dense silt and clayey sand, or soft to firm locally stiff lean clay and silt (N = 4-30)	0-90 ft (210-120 EL) Alternating beds of loose to med-dense silt and clayey sand, or soft to firm locally stiff lean clay and silt (N = 4-30)	No data available
15-25 ft bgs (260-250EL) Beds of med-dense to v.dense silt (N = 24-52) OR Alternating beds of med-dense to dense silt and poorly graded snd with clay (N = 14-41)	55-70 ft (215-200 EL) Alternating beds of dense to v.dense silt and silty sand (N = 42-99)	27-57 ft (238-208 EL) Alternating thin to med beds of med-dense to dense silty sand and clayey sand (N = 14-75)			
25-55 ft bgs (250-220EL) Alternating beds of med-dense to dense poorly graded sand and silty sand (N = 24-38)					

Subsurface soils are expected to be of mostly fluvial origin, becoming lacustrine to the south of Hanford, approaching Tule River.

Valley basement bedrock is generally regard to be greater than 30,000 feet below ground surface for much of the CP2-3 alignment.

5.3 Laboratory Testing

No laboratory testing data are included with the historical Caltrans boring logs.

5.4 Groundwater Levels

A review of historical data nearest piled foundation structures, and expectations of the locations and conditions considered possible in these areas, the design as adopted an assumed groundwater table depth of 5 feet below local ground surface.

5.5 Ground Model

For the interpretation of the very limited historical geotechnical data, a "typical" ground model has been developed for use for all project structures. This ground model, presented in Table 5.5-1, has been assumed to represent a credible geotechnical situation, but not the softest (or stiffest) possible.

By definition, 'worst credible' can be taken to mean that 70% of the geotechnical conditions will be better than the typical ground model and about 30% of conditions will be worse. However it must be pointed out that this definition is applied for convenience only, and in no way implies the existence of data sufficient to convey accuracy or any statistical treatment.

Table 5.5-1
Design Soil Profile for Piled Structures

Soil Design Parameters	Sand A	Sand A	Sand B	Sand C	Sand D	Sand E	Sand F
	(AGWT)	(BGWT)	(BGWT)	(BGWT)	(BGWT)	(BGWT)	(BGWT)
Depth of layer (ft)	0-5	5-10	10-25	25-45	45-65	65-75	>75
N-Value Corrected for Hammer Energy, N_{60} (blows/ft)	10	10	15	10	12	15	50
Friction Angle, ϕ' (deg)	30	30	32	30	31	32	41
Total Unit Weight, γ (pcf)	120	120	120	120	120	120	120
Young's Modulus, E (ksf)	106	106	159	106	127	159	529
Modulus of Horizontal Static Subgrade Reaction, $K_{h,\ static}$ (pci)	58	40	40	40	40	40	80
Modulus of Horizontal Cyclic Subgrade Reaction, $K_{h,\ cyclic}$ (pci)	29	20	20	20	20	20	40

5.6 Liquefaction

Liquefaction risk was discussed in the GSHR.

Because groundwater was assumed to be deeper than 80 feet bgs for CP2-3, liquefaction was generally regarded as not a significant hazard to CP2-3 foundations. This assumption can be applied only for the preparation of reference design. Where piers are anticipated to be within active river channels, were groundwater could be assumed shallower, a minimum additional 20 feet of embedment was assumed to account for the effects of scour. It is assumed that this additional 20 feet of embedment is also sufficient for the liquefaction case in shallow groundwater environments. A design case of liquefaction during extreme scour event was not considered.

The assessment of potentially liquefiable soils will require more rigorous investigation by the design builder to quantify risks and consequences at specific bent locations.

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Section 6.0

Design

6.0 Design

6.1 Pile Design

The magnitude of anticipated axial and lateral column loads drives the assumption that piled foundations are necessary for elevated structures, including overpasses, viaducts, and bridges. Geotechnical input to the design comprised independent single-pile analyses to develop axial, lateral, and rotational spring stiffness. Spring stiffness is provided as a function of applied load to capture non-linear behavior such as lateral deformation.

Implementation of appropriate spring stiffnesses enable structural models to mimic non-linear pile head response related to soil-structure interaction with the assumed ground profile. It is intended that spring stiffness is determined using the tables provided in this section, based on the magnitude of applied load, loading rate (static of cyclic), and pile-cap fixity condition for each foundation pile.

The cases considered for each loading condition are summarized below:

- Lateral Load case
  - Free and fixed pile head
  - Static load and cyclic load soil stiffness
- Bending Moment case
  - Static load and cyclic load soil stiffness
- Vertical Load case
  - Axial stiffness response under (equivalent) static load

Both 6.5- and 9-foot-diameter piles have been evaluated for the above cases, assuming a minimum pile length at least equal to approximately 18 times shaft diameter (115 and 160 feet, respectively). For all analyses, pile head was assumed to begin from 10 feet bgs, to allow for pile cap construction. If a pile would be located in a river channel environment where scour would apply, the structural team assumed the pile head would be 20 feet deeper. Pile spring stiffnesses were determined by performing uncoupled analysis to estimate displacement and rotation at the top of a single pile subject to applied horizontal forces, vertical forces and bending moments. Further details are provided in the subsections that follow.

Limitation of pile head displacement in accordance with project design criteria will provide for a reasonable proportioning of the foundations. While the axial pile stiffness provides an indication of achievable capacity, specific pile bearing capacity estimates and force distribution in pile groups is not included in the scope of this report. For further information on structural modeling and pile design, refer to the main body of this report.

6.1.1.1 Stiffness Response under Lateral Load and Applied Moment

Single pile response to horizontal load and bending moment were evaluated using the geotechnical software LPILE6. The LPILE analysis was undertaken in accordance with the API and Matlock & Reese methods recommended in AASHTO Section A10.2. For simplicity only the short-term concrete modulus was used in calculation; the influence of a long-term concrete modulus is considered negligible by comparison with the uncertainties inherent in more significant assumptions regarding ground conditions.

Both fixed and free pile head conditions were modeled to bound the range of responses possible for variable fixity. The fixed-head rotational stiffness was calculated using an imposed bending moment from

the loading conditions provided by the structural team. Rotational stiffness from applied moment was not calculated for the free-head case.

The results of the analyses are presented in Table 6.1-1 to Table 6.1-6 below.

Table 6.1-1
9ft dia. Pile: Stiffness Matrix of Pile Head Response to Applied Horizontal Load – Free-Head Condition

Horizontal Applied Load (kips)	Pile Head Static Response		Pile Head Cyclic Response	
	$K_{p\text{-}v}$ (kips/in)	$K_{p\text{-}\theta}$ (kips/rad)	$K_{p\text{-}v}$ (kips/in)	$K_{p\text{-}\theta}$ (kips/rad)
0	12,500	-494,600	965	-361,900
200	1,500	-494,600	965	-361,900
400	1,500	-492,900	915	-330,700
600	1,100	-295,600	640	-185,500
800	1,000	-242,500	605	-168,700
1,000	930	-217,500	590	-159,100
1,200	830	-198,700	585	-158,600
1,400	710	-175,200	580	-155,200
1,600	625	-159,900	570	-152,200
1,800	540	-140,800	525	-139,800
2,000	450	-115,500	445	-116,100

Table 6.1-2
9ft dia. Pile: Stiffness Matrix of Pile Head Response to Applied Horizontal Load – Fixed-Head Condition

Horizontal Applied Load (kips)	Pile Head Static Response	Pile Head Cyclic Response
	$K_{p\text{-}v}$ (kips/in)	$K_{p\text{-}v}$ (kips/in)
0	3,555	2,255
200	3,555	2,255
400	3,330	1,825
600	2,620	1,680
800	2,615	1,605
1,000	2,460	1,485
1,200	2,310	1,415
1,400	2,245	1,370
1,600	2,150	1,315
1,800	1,945	1,250
2,000	1,625	1,185

Table 6.1-3
9ft dia. Pile: Stiffness Matrix of Pile Head Response to Applied Bending Moment

Applied Bending Moment (kips x in)	Pile Head Static Response		Pile Head Cyclic Reponse	
	K_{M-y} (kips x in/in)	$K_{M-\theta}$ (kips x in/rad)	K_{M-y} (kips x in/in)	$K_{M-\theta}$ (kips x in/rad)
0	495,000	-90,000,000	362,000	-77,510,000
22,000	495,000	-90,000,000	362,000	-77,510,000
44,000	495,000	-90,000,000	362,000	-77,205,000
66,000	385,000	-42,200,000	278,000	-35,770,000
88,000	260,000	-30,150,000	191,000	-25,770,000
110,000	240,000	-28,350,000	175,000	-24,320,000
132,000	230,000	-27,820,000	165,000	-23,565,000
154,000	224,000	-27,400,000	162,000	-23,285,000
176,000	220,000	-27,020,000	159,000	-23,005,000
198,000	213,000	-25,400,000	155,000	-21,965,000
220,000	202,000	-22,400,000	148,000	-19,270,000

Table 6.1-4
6.5ft dia. Pile: Stiffness Matrix of Pile Head Response to Applied Horizontal Load – Free-Head Condition

Horizontal Applied Load (kips)	Pile Head Static Response		Pile Head Cyclic Response	
	K_{p-y} (kips/in)	$K_{p-\theta}$ (kips/rad)	K_{p-y} (kips/in)	$K_{p-\theta}$ (kips/rad)
0	1,000	-244,000	630	-177,000
50	1,000	-244,000	630	-177,000
100	1,000	-244,000	630	-177,000
150	1,000	-244,000	630	-177,000
200	1,000	-244,000	630	-177,000
250	1,000	-241,000	510	-125,000
300	805	-167,000	445	-100,000
350	727	-142,000	420	-91,000
400	685	-127,000	405	-85,000
450	660	-120,000	400	-84,000
500	615	-110,000	395	-82,000

Table 6.1-5
6.5ft dia. Pile: Stiffness Matrix of Pile Head Response to Applied Horizontal Load – Fixed-Head Condition

Horizontal Applied Load (kips)	Pile Head Static Response	Pile Head Cyclic Response
	K_{p-y} (kips/in)	K_{p-y} (kips/in)
0	2,300	1,450
50	2,300	1,450
100	2,300	1,450
150	2,300	1,450
200	2,300	1,200
250	1,900	1,100
300	1,700	1,100
350	1,700	1,050
400	1,700	1,050
450	1,700	1,000
500	1,650	990
1,200	-	710
1,520	-	495
1,700	-	330

Table 6.1-6
6.5ft dia. Pile: Stiffness Matrix of Pile Head Response to Applied Bending Moment

Applied Bending Moment (kips x in)	Pile Head Static Response		Pile Head Cyclic Response	
	K_{M-y} (kips x in/in)	$K_{M-\theta}$ (kips x in/rad)	K_{M-y} (kips x in/in)	$K_{M-\theta}$ (kips x in/rad)
0	245,000	-33,140,000	178,000	-28,270,000
5,000	245,000	-33,140,000	178,000	-28,270,000
10,000	245,000	-33,065,000	178,000	-28,208,000
15,000	245,000	-32,990,000	178,000	-28,140,000
20,000	245,000	-32,910,000	178,000	-28,075,000
25,000	219,000	-19,360,000	156,000	-15,905,000
30,000	148,000	-12,235,000	111,000	-10,695,000
35,000	133,000	-11,275,000	96,000	-9,590,000
40,000	126,000	-10,880,000	89,000	-9,135,000

45,000	120,000	-10,570,000	86,000	-8,965,000
50,000	118,000	-10,435,000	84,000	-8,825,000
107,700	-	-	63,400	"
114,600	-	-	37,250	"
117,100	-	-	22,650	"

" Similar value as previous increment has been assumed.

6.1.1.2 Stiffness Response under Axial Load

In estimation of single pile axial stiffness, two different approaches were implemented. The first method employs a methodology proposed by Fleming in the paper "A new method for single pile settlement prediction and analysis", 1992, Geotechnique 42, No. 3, pp. 411-425. The second method is based on methodology proposed by Reese, Isenhower, and Wang in "Analyses and design of shallow and deep foundations", 2006. Both methods model the settlement of a single pile under incremental vertical loading to develop a vertical stiffness.

The lower stiffness of the two responses has been adopted, generally derived from the Reese et al method. The variation of axial stiffness with increasing vertical load was observed to be minimal for vertical loads within the range anticipated for design. Therefore, only a single axial stiffness has been provided in Table 6.1-7 for each of the pile sizes evaluated.

Table 6.1-7
Vertical Stiffness under Axial Load

Pile size	K_{t-z} (kips/in)
9-ft dia.	10,750 (for up to ~4,100 kips applied)
6.5-ft dia.	4,840 (for up to ~ 2,000 kips applied)

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Section 7.0

Limitations and Further Information

7.0 Limitations and Further Information

The Procurement Package design effort for Construction Package 2-3 is based on extremely limited information included in historical geotechnical reports. As a consequence, there may be significant changes necessary in detailed design. The results of this report should be considered preliminary and refined by the design-build teams during final design once site-specific information is available.

As discussed in Section 1.0, a limited but project-specific geotechnical investigation has recently been completed along the CP2-3 corridor; however this information has not been used in the reference design or in generating the parameters and recommendations of this Appendix. The impression of ground conditions provided by a brief review of this recent data suggests it is reasonable to expect Fresno County, on average, to be fairly represented by the advice provided in this document for reference design purposes. Select geotechnical explorations in Tulare County suggest that some local areas may differ considerably from these assumptions.

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Section 8.0

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Appendix B

Seismic Analysis Design Plan

APPENDIX B Seismic Analysis Design Plan

Attached is the Seismic analysis design Plan which was prepared in order to agree the necessary analyses which are the subject of this report.

California High-Speed Rail Project Engineering

Complex and Non-Standard Structures Seismic Analysis Plan

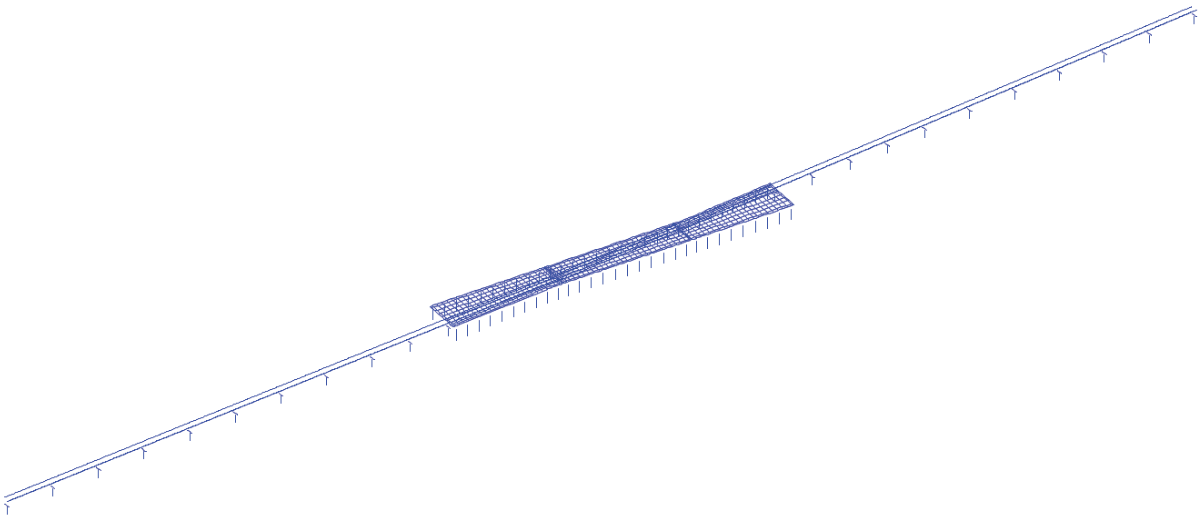
RFP No.: 13-57 – Addendum No. 2 - 06/30/2014

Prepared by:

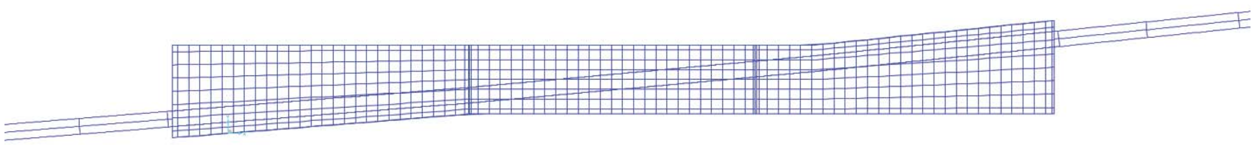
URS/HMM/Arup Joint Venture

October 2012

1. **Package 2: H Conejo Crossover over BNSF**



Conejo: 3D View



Conejo: Plan View

- The 950-foot crossover structure that crosses two existing BNSF tracks consists of 32no. 30-foot bays. The span is located between bents 17 and 18 of the Conejo BNSF Viaduct. The height of the structure is about 44 feet, measured from top of OG to top of rail. The span in the transverse direction varies from 74’ to 101’ between columns centerlines. Columns are 8’ diameter and edge beams are 12’ deep by 10’ wide. The 6” thick concrete deck slab is assumed to be composite with 5’-6” deep precast cross beams spanning the transverse direction. Precast cross beams are spaced at 4’ centers, leaving no clear space between adjacent beams. The beams are seated on the edge beam and cast-in to form a full moment connection.
- Joints are located in the structure to accommodate the requirement of 330’ maximum thermal unit lengths. The crossover thermal units and standard viaduct spans immediately adjacent to the crossover structure are connected with dowel elements. These connections allow longitudinal and vertical movement but restrict relative transverse displacements between units. The dowels are aligned with the HST track axes to ensure longitudinal movements are in the plane of the rails.
  Themal unit lengths: 310’, 320’, 310’
- Crossover foundations are made up of 10’x4’ barrettes, situated beneath each column. The columns supporting the standard viaduct immediately adjacent to crossover structure will be founded by 2no. 6.5’ diameter piles. The remaining standard viaduct columns are to be founded by 4no.6.5’ diameter piles.

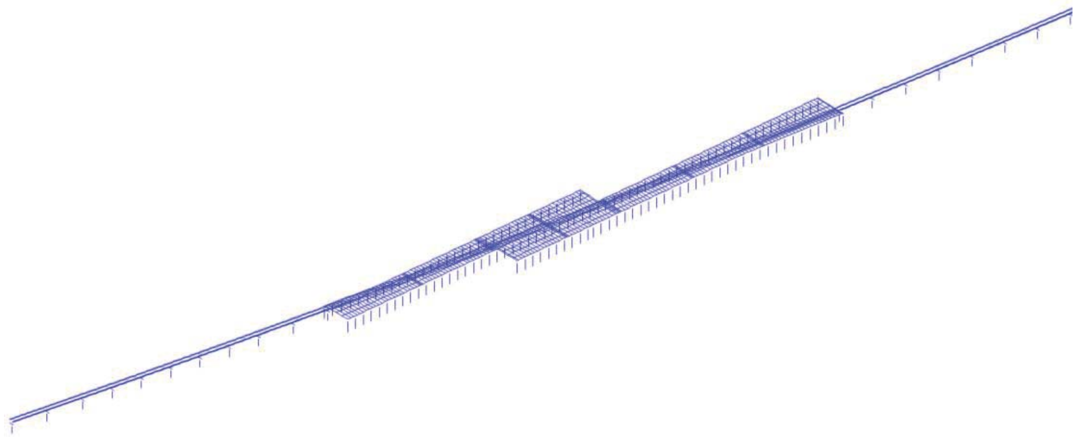
- General Classification: Primary Structure (HST Bridge, TM 2.10.4 Section 6.5.1.1)
    - Design life is 100 years.
    - Seismic design must comply with TM 2.10.4.
    - When applying the AASHTO LRFD code, values for the importance, ductility, and redundancy factors, h_I, h_D, and h_R, have been chosen as
        - Operational factor h_I = 1.05 for critical or essential bridges
        - Ductility factor h_D =1.05 for non-ductile components strength limit states; 1.00 for conventional designs and details
        - Redundancy factor h_R = 1.05 for non-redundant elements, 1.00 otherwise
- Importance Classification: Important Structure (TM 2.10.4 Section 6.5.1.2)
- Technical Classification: Complex Structure (Irregular Geometry, TM 2.10.4 Section 6.5.1.3)
- Bridge skew: 0° to 5°
- Frequency Analysis Check Limits:
  Span of bridge is defined in the transverse direction, as it is the dominant mode for vertical frequency. The portal frame definition is used, i.e. heights of columns are added as two additional spans, according to TM 2.10.10 Section 6.8.2.

  At longest span (100ft):
  L_{avg} = (35.5+101+35.5)/3 ft = 57.3 ft, k = 1.3, L = kL_{avg} = 74.5 ft
  Vertical: $\eta_{lower} = 47.645L^{-0.592} = 3.71$ Hz;
  $\eta_{upper} = 230.46L^{-0.748} = 9.17$ Hz
  Transverse: $\eta_{trans} > 1.2$ Hz
  Longitudinal $\eta_{torsion} > 1.2\eta_{vert}$

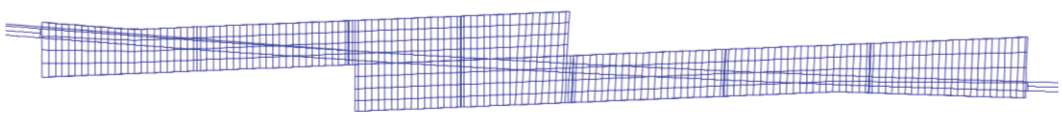
  At shortest span (74ft):
  L_{avg} = (35.5+74+35.5)/3 ft = 48.3 ft, k = 1.3, L = kL_{avg} = 62.8 ft
  Vertical: $\eta_{lower} = 47.645L^{-0.592} = 4.11$ Hz;
  $\eta_{upper} = 230.46L^{-0.748} = 10.4$ Hz
  Transverse: $\eta_{trans} > 1.2$ Hz
  Longitudinal $\eta_{torsion} > 1.2\eta_{vert}$

- Seismic Performance Criteria:
  Elastic structural response under OBE; No collapse under MCE
- Ground Motion Zone: 4
- I of three beams + 6”" slab (12' wide) = 136 ft^4; I of crossbeams per 30' = 340 ft^4; I of Dia 8' column = 224 ft^4 -> OK for strong beam weak column
- Analysis approach:
    - Non-linear time-history analysis for OBE events for structure design
    - Non-linear time-history analysis for Track-Structure Interaction analysis;
    - Displacement demand from nonlinear time history analysis for MCE events.

2. Package 3: C2 Corcoran Crossover over BNSF and SR43



Corcoran: 3D View



Corcoran: Plan View

- The 1918-foot crossover structure that crosses Central Valley Highway / State Route 43 consists of 64no. 30-foot bays. The span is located between bents 32 and 33 of the C2 State Route 43 BNSF Viaduct. The height of the structure is about 44.5 feet, measured from top of OG to top of rail. The span in the transverse direction varies from 80’ to 105’ between columns centerlines. Columns are 8’ diameter and edge beams are 12’ deep by 10’ wide. The 6” thick concrete deck slab is assumed to be composite with 5’-6” deep precast cross beams spanning the transverse direction.
- Joints are located in the structure to accommodate the requirement of 330’ maximum thermal unit length. The crossover thermal units and standard viaduct spans immediately adjacent to the crossover structure are connected with dowel elements. These connections allow longitudinal and vertical movement but restrict relative transverse displacements between units. The dowels are aligned with the HST track axes to ensure longitudinal movements are in the plane of the rails.
- Crossover foundations are made up of 10’x4’ barrettes, situated beneath each column. The columns supporting the standard viaduct immediately adjacent to crossover structure will be founded by 2no. 6.5’ diameter piles. The remaining standard viaduct columns are to be founded by 4no.6.5’ diameter piles.
- Thermal Unit lengths: 318’, 300’, 204’, 204’, 300’, 300’, 287’
- General Classification: Primary Structure (HST Bridge, TM 2.10.4 Section 6.5.1.1)
  - Design life is 100 years.
  - Seismic design must comply with TM 2.10.4.
  - When applying the AASHTO LRFD code, values for the importance, ductility, and redundancy factors, h_I, h_D, and h_R, have been chosen as
    - Operational factor h_I = 1.05 for critical or essential bridges
    - Ductility factor h_D =1.05 for non-ductile components strength limit states; 1.00 for conventional designs and details

    - Redundancy factor h_R = 1.05 for non-redundant elements, 1.00 otherwise

- Importance Classification: Important Structure (TM 2.10.4 Section 6.5.1.2)
- Technical Classification: Complex Structure (Irregular Geometry, TM 2.10.4 Section 6.5.1.3)
- Bridge skew: 0° to 5°
- Frequency Analysis Check Limits:
  Span of bridge is defined in the transverse direction, as it is the dominant mode for vertical frequency. The portal frame definition is used, i.e. height of columns are added as two additional spans, according to TM 2.10.10 Section 6.8.2.

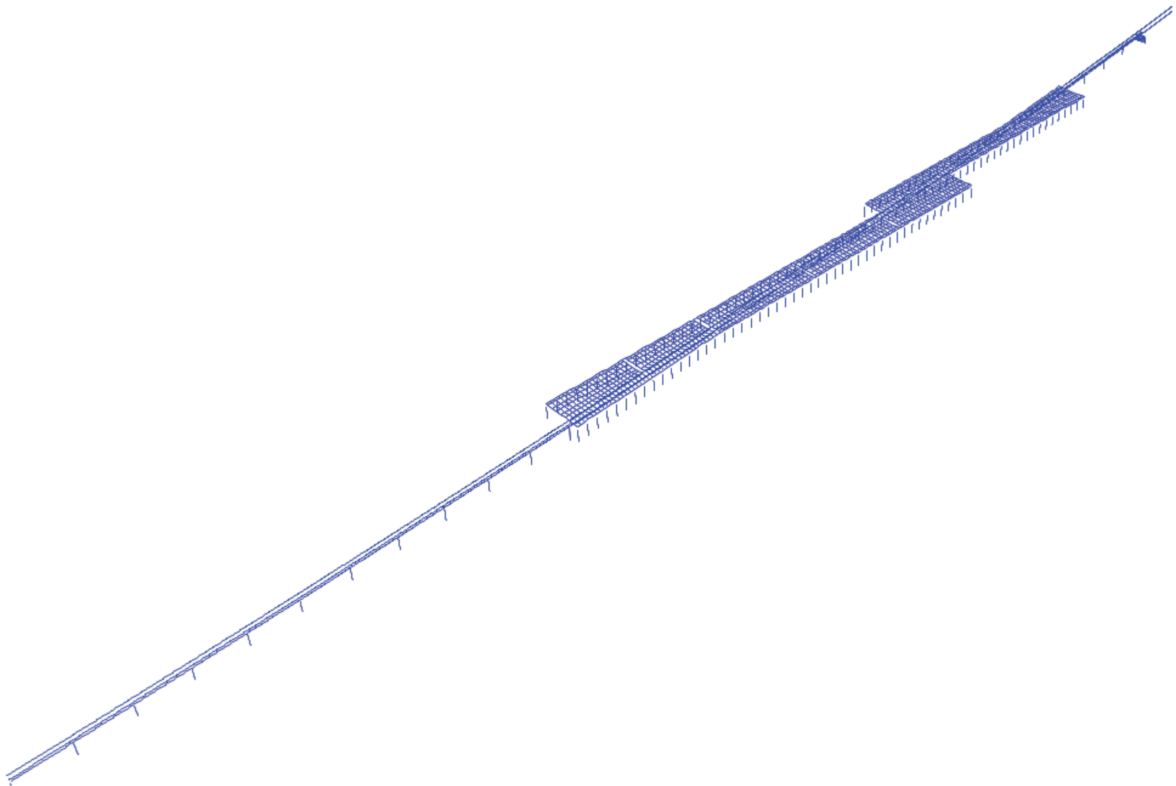
  At Single Span L = 115’ maximum average span at segment:
  L_{avg} = (41+115+41)/3 ft = 65.7 ft, k = 1.3, L = kL_{avg} = 85 ft
  Vertical: $\eta_{lower} = 47.645L^{-0.592}$ = 3.43 Hz;
  $\eta_{upper} = 230.46L^{-0.748}$ = 8.28 Hz
  Transverse: η_{trans} > 1.2 Hz
  Longitudinal $\eta_{torsion} > 1.2\eta_{vert}$

  At Single Span L=80’ minimum average span at segment:
  L_{avg} = (41+80+41)/3 ft = 54 ft, k = 1.3, L = kL_{avg} = 70.2 ft
  Vertical: $\eta_{lower} = 47.645L^{-0.592}$ = 3.85 Hz;
  $\eta_{upper} = 230.46L^{-0.748}$ = 9.58 Hz
  Transverse: η_{trans} > 1.2 Hz
  Longitudinal $\eta_{torsion} > 1.2\eta_{vert}$

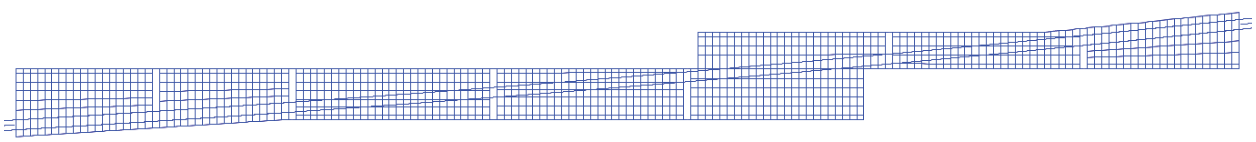
  At Multi-Span L1=80’, L2=85’:
  L_{avg} = (41+80+85+41)/4 ft = 61.75 ft, k = 1.4, L = kL_{avg} = 86.45 ft
  Vertical: $\eta_{lower} = 47.645L^{-0.592}$ = 3.40 Hz;
  $\eta_{upper} = 230.46L^{-0.748}$ = 8.20 Hz
  Transverse: η_{trans} > 1.2 Hz
  Longitudinal $\eta_{torsion} > 1.2\eta_{vert}$

- Seismic Performance Criteria:
  Elastic structural response under OBE; No collapse under MCE
- Ground Motion Zone: 5
- I of three beams + 6”" slab (12' wide) = 136 ft^4; I of crossbeams per 30' = 340 ft^4; I of Dia 8' column = 224 ft^4 -> OK for strong beam weak column
- Analysis approach:
  - Non-linear time-history analysis for OBE events for structure design
  - Non-linear time-history analysis for Track-Structure Interaction analysis;
  - Displacement demand from nonlinear time history analysis for MCE events.

3. Package 4: L4 Poso Creek Crossover over BNSF and SR43



Poso: 3D View



Poso: Plan View

- The 2180-foot crossover structure that crosses Central Valley Highway / State Route 43 consists of 68no. 30-foot bays. The span is located between bents 35 and 36 of the L4 Poso Creek BNSF Viaduct. The height of the structure varies between 32 and 41 feet, measured from top of OG to top of rail. The center to center column spans in the transverse direction varies from 60’ to 114’. Columns are 8’ diameter and edge beams are 12’ deep by 10’ wide. The 6” thick concrete deck slab is assumed to be composite with 5’-6” deep precast cross beams spanning the transverse direction.
- Joints are located in the structure to accommodate the requirement of 330’ maximum thermal unit length. The crossover thermal units and standard viaduct spans immediately adjacent to the crossover structure are connected with dowel elements. These connections allow longitudinal and vertical movement but restrict relative transverse displacements between units. The dowels are aligned with the HST track axes to ensure longitudinal movements are in the plane of the rails.
  Thermal unit lengths: 232’, 232’, 330’, 330’, 330’, 330’, 265’

- Crossover foundations are made up of 10’x4’ barrettes, situated beneath each column. The columns supporting the standard viaduct immediately adjacent to crossover structure will be founded by 2no. 6.5’ diameter piles. The remaining standard viaduct columns are to be founded by 4no.6.5’ diameter piles.
- General Classification: Primary Structure (HST Bridge, TM 2.10.4 Section 6.5.1.1)
    - Design life is 100 years.
    - Seismic design must comply with TM 2.10.4.
    - When applying the AASHTO LRFD code, values for the importance, ductility, and redundancy factors, h_I, h_D, and h_R, have been chosen as
        - Operational factor h_I = 1.05 for critical or essential bridges
        - Ductility factor h_D =1.05 for non-ductile components strength limit states; 1.00 for conventional designs and details
        - Redundancy factor h_R = 1.05 for non-redundant elements, 1.00 otherwise
- Importance Classification: Important Structure (TM 2.10.4 Section 6.5.1.2)
- Technical Classification: Complex Structure (Irregular Geometry, TM 2.10.4 Section 6.5.1.3)
- Bridge skew: 4.5°
- Frequency Analysis Check Limits:
  Span of bridge is defined in the transverse direction, as it is the dominant mode for vertical frequency. The portal frame definition is used, i.e. height of columns are added as two additional spans, according to TM 2.10.10 Section 6.8.2.

  At Single Span L = 106’ maximum average span at segment:
  L_{avg} = (37+106+37)/3 ft = 60 ft, k = 1.3, L = kL_{avg} = 78 ft
  Vertical: $\eta_{lower} = 47.645L^{-0.592} = 3.61$ Hz;
  $\eta_{upper} = 230.46L^{-0.748} = 8.86$ Hz
  Transverse: $\eta_{trans} > 1.2$ Hz
  Longitudinal $\eta_{torsion} > 1.2\eta_{vert}$

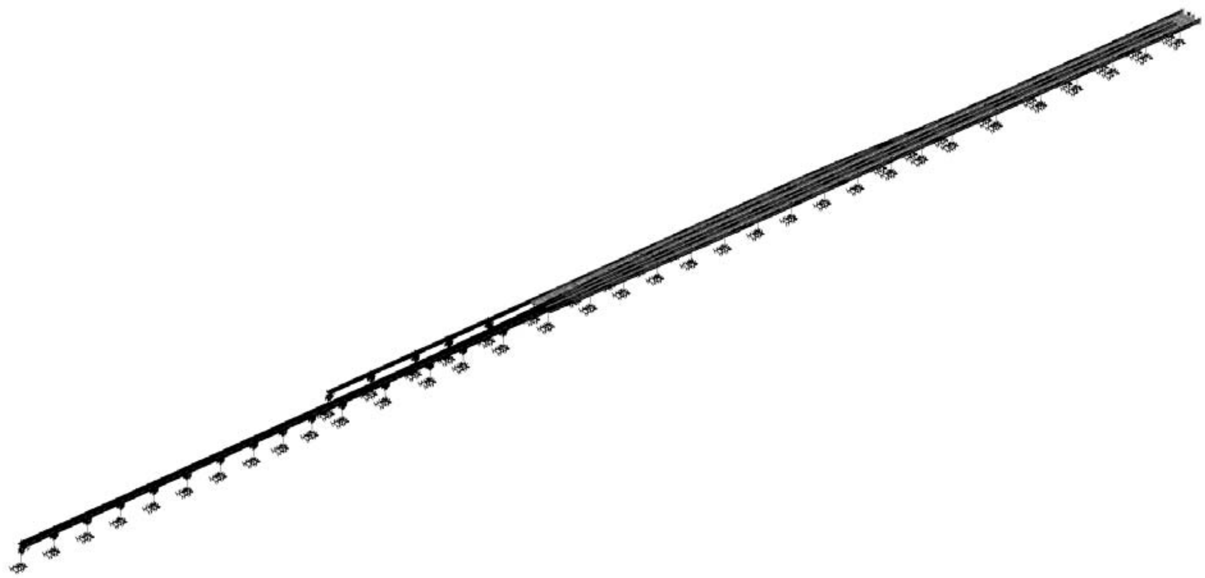
  At Single Span L=60’ minimum average span at segment:
  L_{avg} = (37+60+37)/3 ft = 44.7 ft, k = 1.3, L = kL_{avg} = 58.1 ft
  Vertical: $\eta_{lower} = 262.5/L = 4.52$ Hz;
  $\eta_{upper} = 230.46L^{-0.748} = 11.05$ Hz
  Transverse: $\eta_{trans} > 1.2$ Hz
  Longitudinal $\eta_{torsion} > 1.2\eta_{vert}$

  At Multi-Span L1=60’, L2=85’:
  L_{avg} = (39+60+85+39)/4 ft = 55.75 ft, k = 1.4, L = kL_{avg} = 78.05 ft
  Vertical: $\eta_{lower} = 47.645L^{-0.592} = 3.61$ Hz;
  $\eta_{upper} = 230.46L^{-0.748} = 8.85$ Hz
  Transverse: $\eta_{trans} > 1.2$ Hz
  Longitudinal $\eta_{torsion} > 1.2\eta_{vert}$

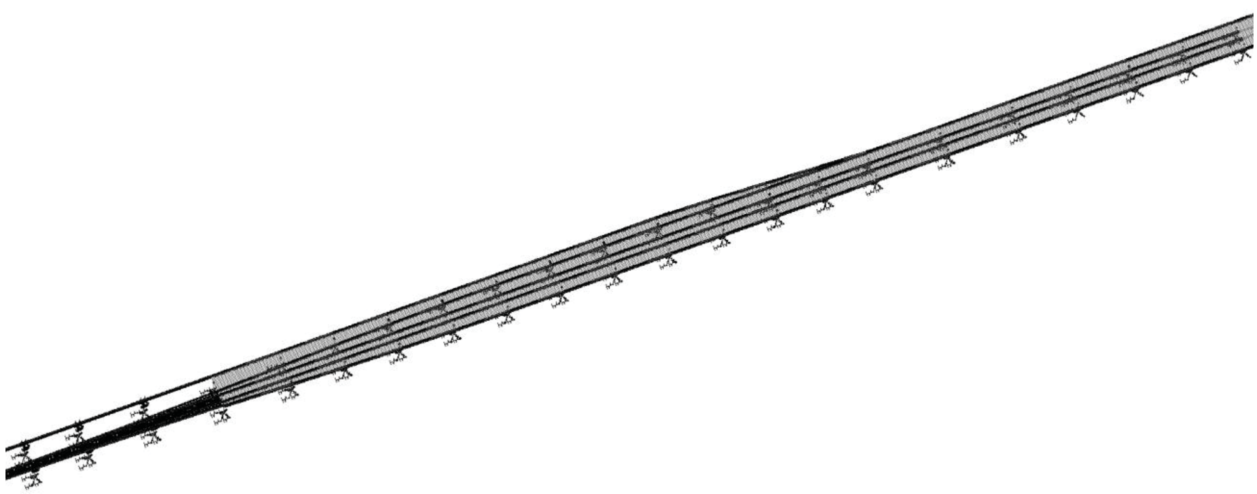
- Seismic Performance Criteria:
  Elastic structural response under OBE; No collapse under MCE
- Ground Motion Zone: 6
- I of three beams + 6”" slab (12' wide) = 136 ft^4; I of crossbeams per 30' = 340 ft^4; I of Dia 8' column = 224 ft^4 -> OK for strong beam weak column
- Analysis approach:

- Non-linear time-history analysis for OBE events for structure design
- Non-linear time-history analysis for Track-Structure Interaction analysis;
- Displacement demand from nonlinear time history analysis for MCE events.

4. Package 2:Hanford Station



Hanford Station: 3D View (Half of the symmetric structure)



Hanford Station: Plan View (Half of the symmetric structure)

- Hanford Station is a 6000-foot long aerial structure. At both ends of the station, two 100mph turnouts are located to connect the platform tracks to the through tracks. The structure accommodates a total of 4 tracks in the station. In addition there are two refuge tracks which connect via 50 mph turnouts to the each platform track next to the platform structure. The station layout is therefore rotationally symmetric about the center point. The height of the structure varies from about 40 to 47 feet, measured from top of OG to top of rail. The maximum width of the structures is 115ft where platforms are located and it is narrowed down beyond the end of the platforms. Columns are 8' (to be verified) diameter.

- Joints are located in the structure to accommodate the requirement of 330’ maximum thermal unit lengths. The locations of these joints were determined to avoid the area of special track supporting plates and within the vicinity of the movable portions of switches and frogs as set out in a Memo dated October 3rd 2012. The structural spans within the station area vary, thermal lengths are consequently: 110’, 120’, 130’, 150’, 160’, 265’ & 315’
- The columns for the aerial structure are founded on 8ft thick 39’x39’ pile caps each supported by four 6.5’ diameter piles.
- General Classification: Primary Structure (HST Bridge, TM 2.10.4 Section 6.5.1.1)
    - Design life is 100 years.
    - Seismic design must comply with TM 2.10.4.
    - When applying the AASHTO LRFD code, values for the importance, ductility, and redundancy factors, h_I, h_D, and h_R, have been chosen as
        - Operational factor h_I = 1.05 for critical or essential bridges
        - Ductility factor h_D =1.05 for non-ductile components strength limit states; 1.00 for conventional designs and details
        - Redundancy factor h_R = 1.05 for non-redundant elements, 1.00 otherwise
- Importance Classification: Important Structure (TM 2.10.4 Section 6.5.1.2)
- Technical Classification: Complex Structure (Irregular Geometry, TM 2.10.4 Section 6.5.1.3)
- Bridge skew: 0° (although turnout tracks may cross joints at a small angle)
- Frequency Analysis Check Limits:
  Frequency limits were determined according to TM 2.10.10 Section 6.8.2.

  At longest span (160ft):
  L_{avg} = (155+160)/2 ft = 157.5 ft, k = 1.2, L = kL_{avg} = 189 ft
  Vertical: $\eta_{lower} = 47.645L^{-0.592} = 2.14$ Hz;
  $\eta_{upper} = 230.46L^{-0.748} = 4.57$ Hz
  Transverse: $\eta_{trans} > 1.2$ Hz
  Longitudinal $\eta_{torsion} > 1.2\eta_{vert}$ Hz

  At shortest span (110ft):
  L = 110 ft simple span
  Vertical: $\eta_{lower} = 47.645L^{-0.592} = 2.95$ Hz;
  $\eta_{upper} = 230.46L^{-0.748} = 6.85$ Hz
  Transverse: $\eta_{trans} > 1.2$ Hz
  Longitudinal $\eta_{torsion} > 1.2\eta_{vert}$ Hz

- Seismic Performance Criteria:
  Elastic structural response under OBE; No collapse under MCE
- Ground Motion Zone: 4
- I of two girders = 2 × 585 ft^4 = 1170 ft^4 ; I of Dia 8' column = 224 ft^4 → OK for strong beam weak column
- Analysis approach:
    - Non-linear time-history analysis for OBE events for structure design
    - Non-linear time-history analysis for Track-Structure Interaction analysis;
    - Thermal displacements
    - Displacement demand from nonlinear time history analysis for MCE events.

5. General Analysis Approach

Analysis approach for OBE Strength Design

- Strength 5 Load Combination: 1.25 DC + 1.5 DW + 0.5 ($LLRR_1$+IM_1+CF_1+LF_1) + 1.0 OBE
- Non-linear Time Histories:
  two horizontal directions only, data provided by EMT in 7 sets for Zones 4-6 with 5% damping. Data pairs in each set to be applied Longitudinal/Transverse and Transverse/Longitudinal.
- No time lag is incorporated in the analysis for non-linear time history analysis as there is insufficient ground investigation data to develop an accurate assumption for time lag.
- Seismic capacities for OBE: ϕF_N = 0.9 in accordance with CBDS

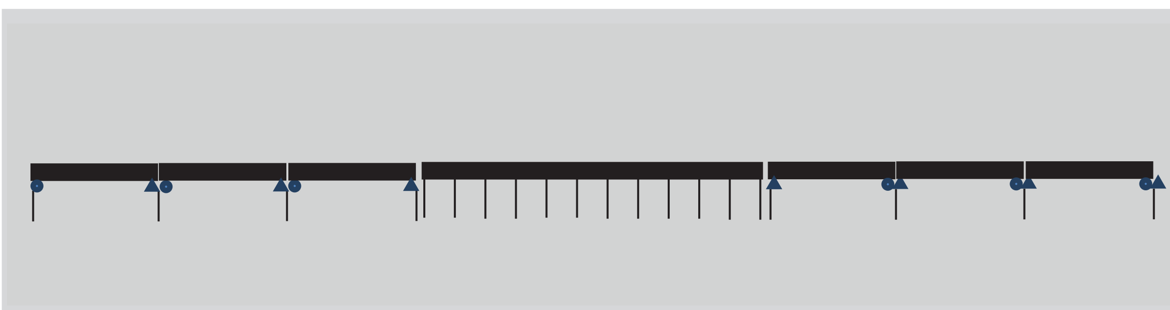
Analysis approach for MCE Strength Design

- Extreme 3 Load Combination: 1.0 (DC+DW) + 1.0 MCE
- Non-linear Time Histories:
  two horizontal directions only, data provided by EMT in 7 sets for Zones 4-6 with 5% damping. Data pairs in each set to be applied Longitudinal/Transverse and Transverse/Longitudinal.
- No time lag is incorporated in the analysis for non-linear time history analysis as there is insufficient ground investigation data to develop an accurate assumption for time lag.

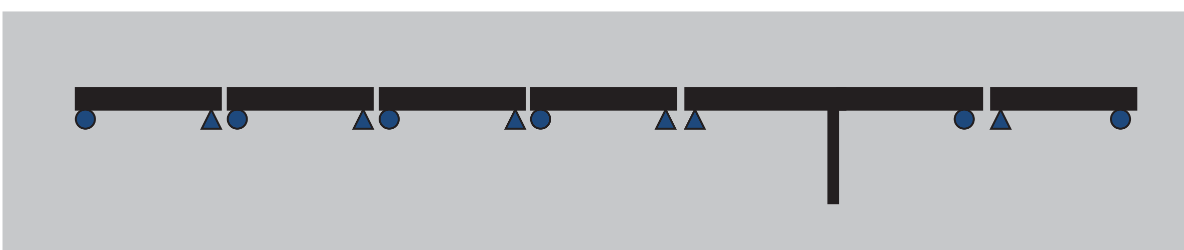
Software used for analysis.

- SAP2000 v.15

Modeling assumptions



Articulation (Conejo Crossover, Corcoran Crossover, Poso Creek Crossover)

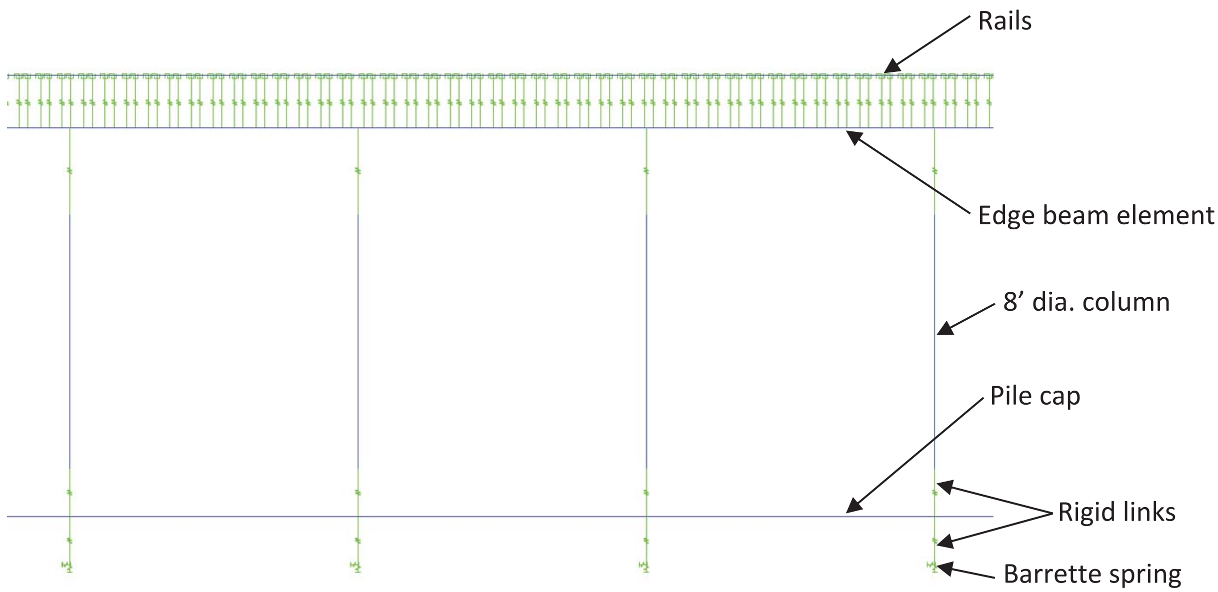


Articulation (Hanford Station)

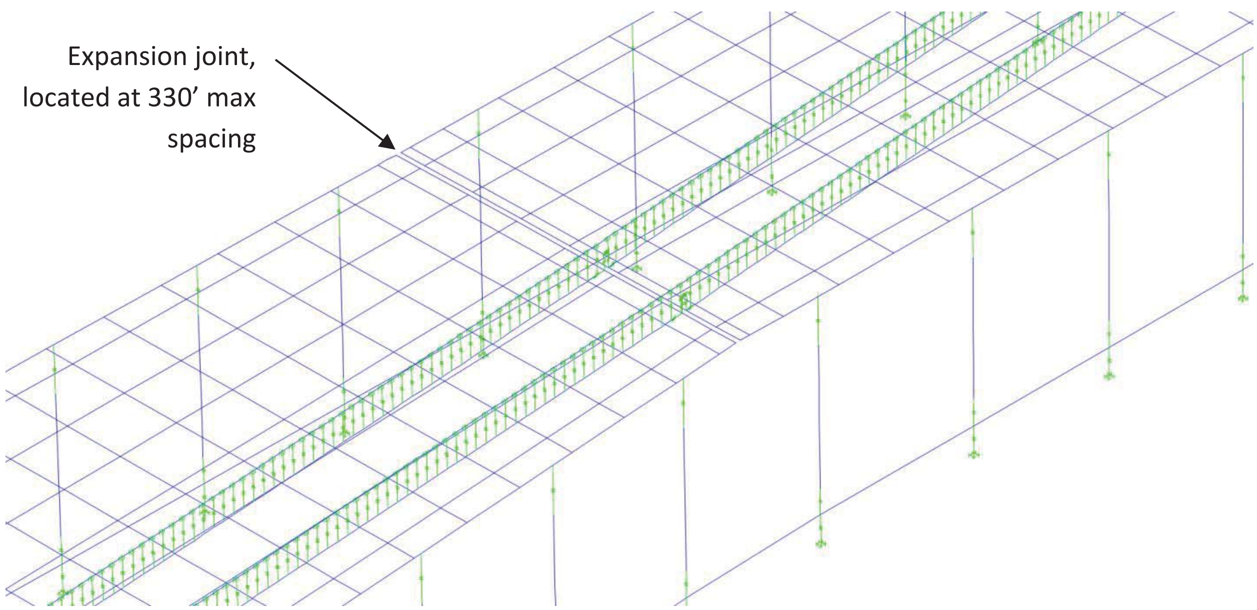
- Model 1 for design of H Conejo Crossover:
  10 standard spans, crossover structure, 10 standard spans
- Model 2 for design of C2 Corcoran Crossover:
  10 standard spans, crossover structure, 10 standard spans
- Model 3 for design of L4 Poso Creek Crossover:
  10 standard spans, crossover structure, 4 standard spans, south abutment with rails and fasteners extended

- Model for design of Hanford Station
  10 standard spans, half of Hanford station structure
- The same models (with different mass and stiffness modifiers) will be used for both non-linear time-history analysis of seismic structure design and train-structure interaction analyses
- Non-linear time-history analysis will use the "modal analysis" option in SAP2000 to limit the analysis time required. One "direct integration" case will be run to compare with results from the equivalent "modal" case.
  Generally investigations have shown that the "modal" method gives results that are about 30% higher for rail stress than the "direct integration" (exact) option.
  For maximum displacement of joints, the "modal" method gives results that are about 10% to 20% more conservative than direct integration, but maximum relative displacements between joints are about 10% less conservative. For this level of design it is appropriate to be conservative so the above percentages will be considered in the assessment of results.
- The structure model will be a "Stick" model for columns and standard superstructure. The deck of crossover sections will be modeled as stick elements in the transverse direction, but held by longitudinal elements to simulate the longitudinal stiffness of the deck.
- On the structures, the rail elements and fasteners are connected to the superstructure by rigid links.
- A longitudinal spring of stiffness 24200 k/ft is added at the dead end of each track on both ends per TM. (TM 2.10.10 R1, Section 6.13.7)
- The foundations are modeled by a spring matrix for each individual pile. The pile cap will be modeled as rigid links connecting top of barrettes/piles to bottom of columns. The spring matrix has been derived from historic ground investigation results and uses the program LPILE to simulate the performance of the soil/pile system.
- The displacement capacity will be calculated using SAP and confirmed with hand calculations
- Loading DW is added on structures;
  LLRM, IM, CF, and LF are added on rail elements

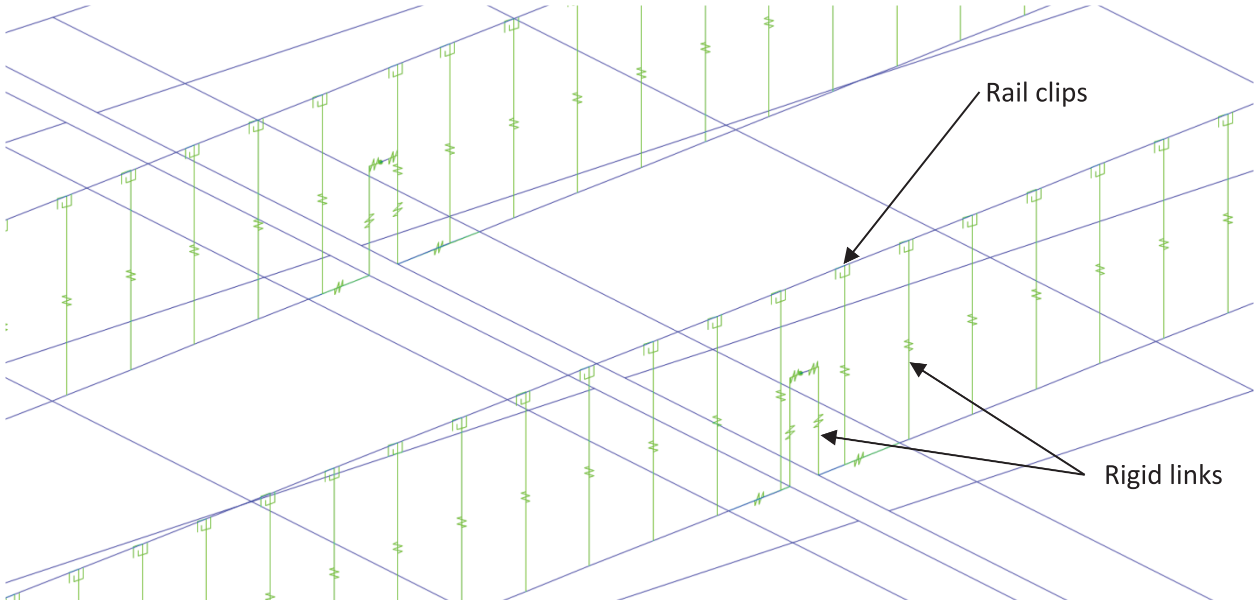
- SAP Model layout: Conejo Crossover, Corcoran Crossover, Poso Creek Crossover



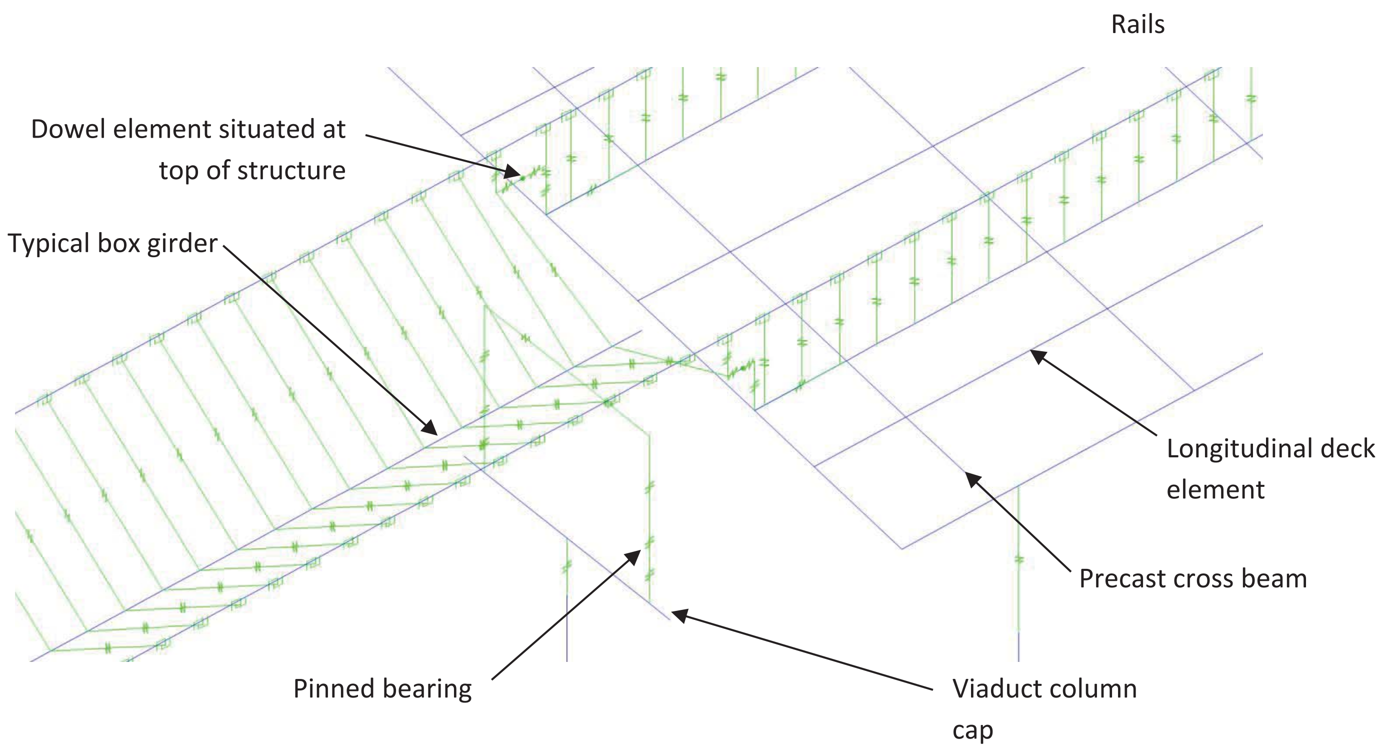
SAP Model: Elevation



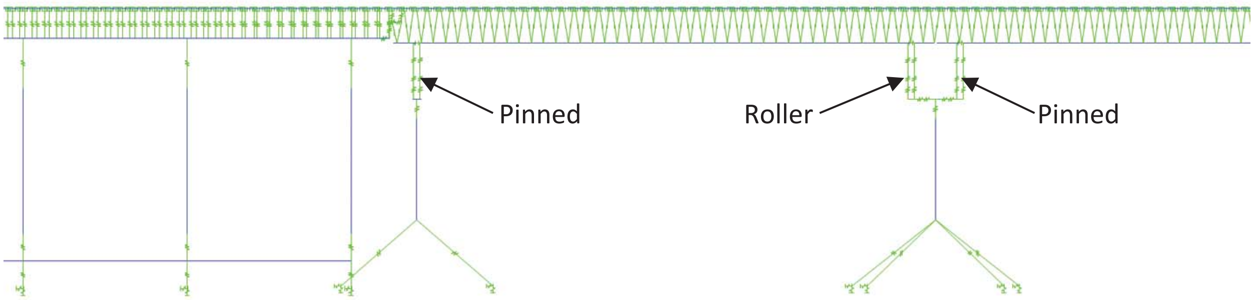
SAP Model: 3D View Internal Joint



SAP Model: Internal Joint Detail

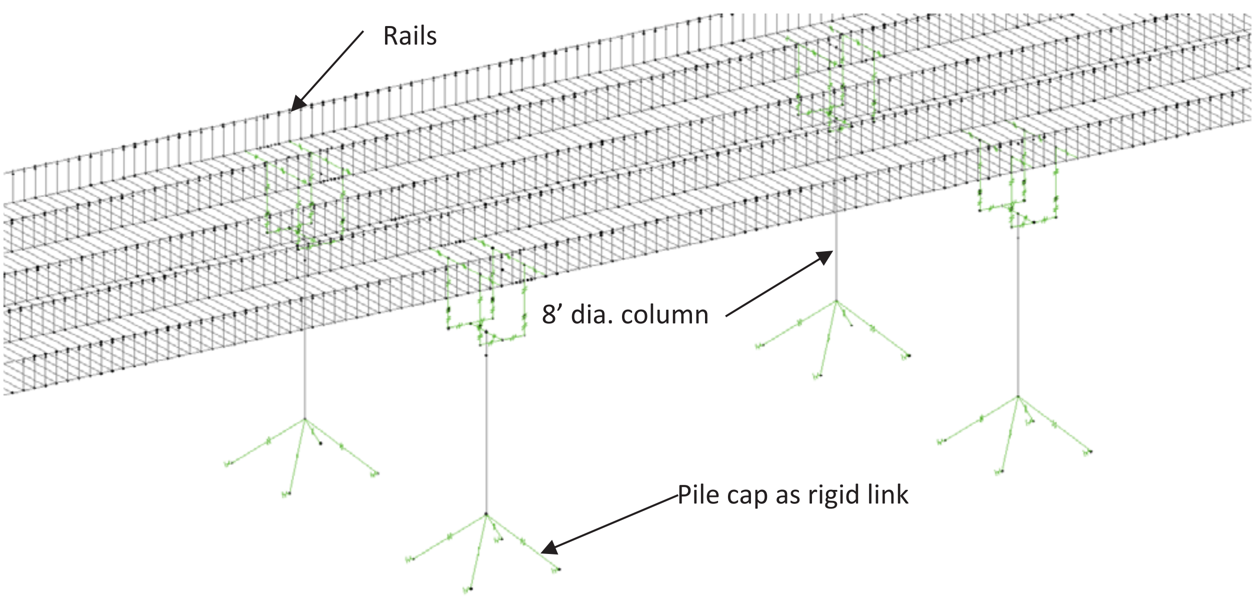


SAP Model: Transition Joint Detail

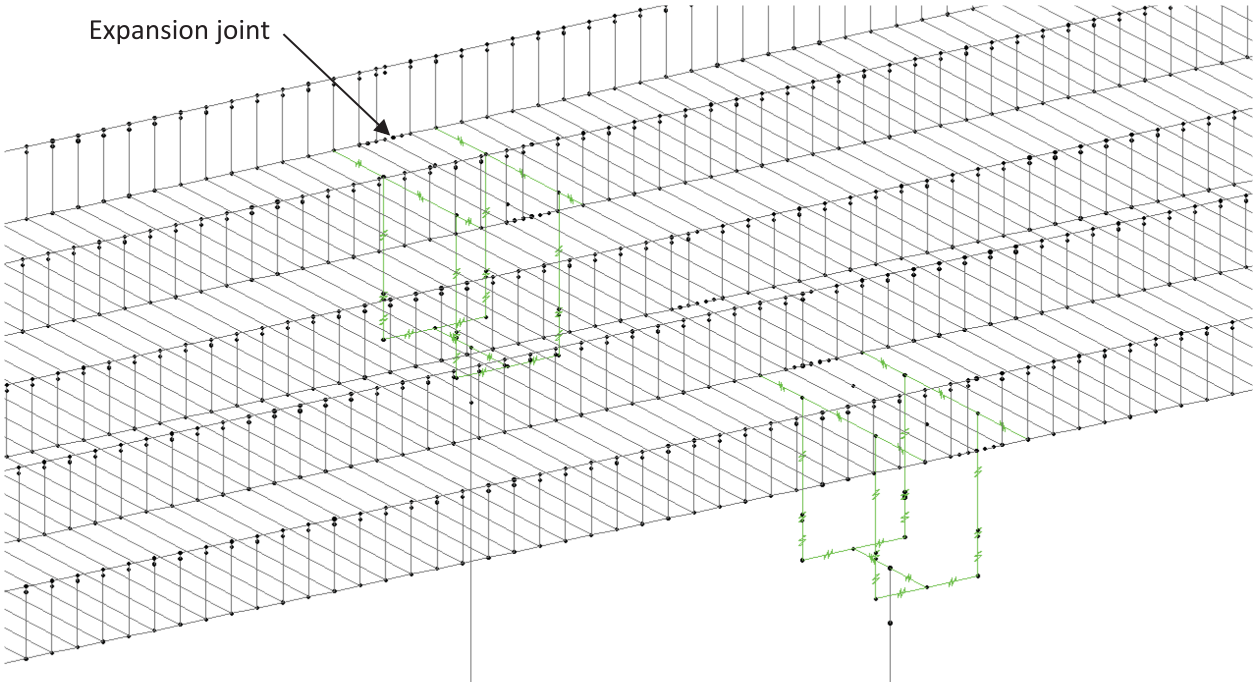


SAP Model: Viaduct Articulation

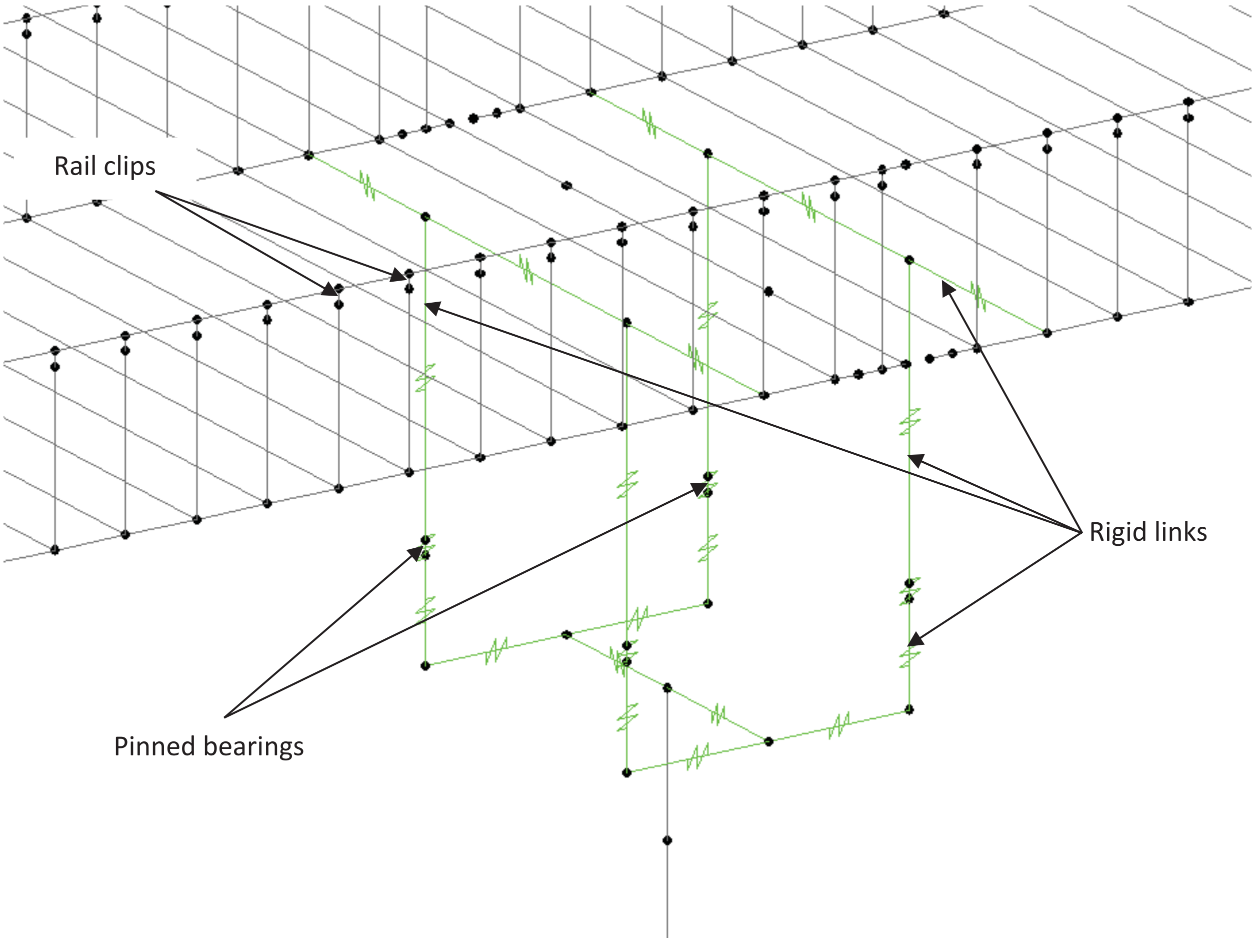
- SAP Model layout: Hanford Station



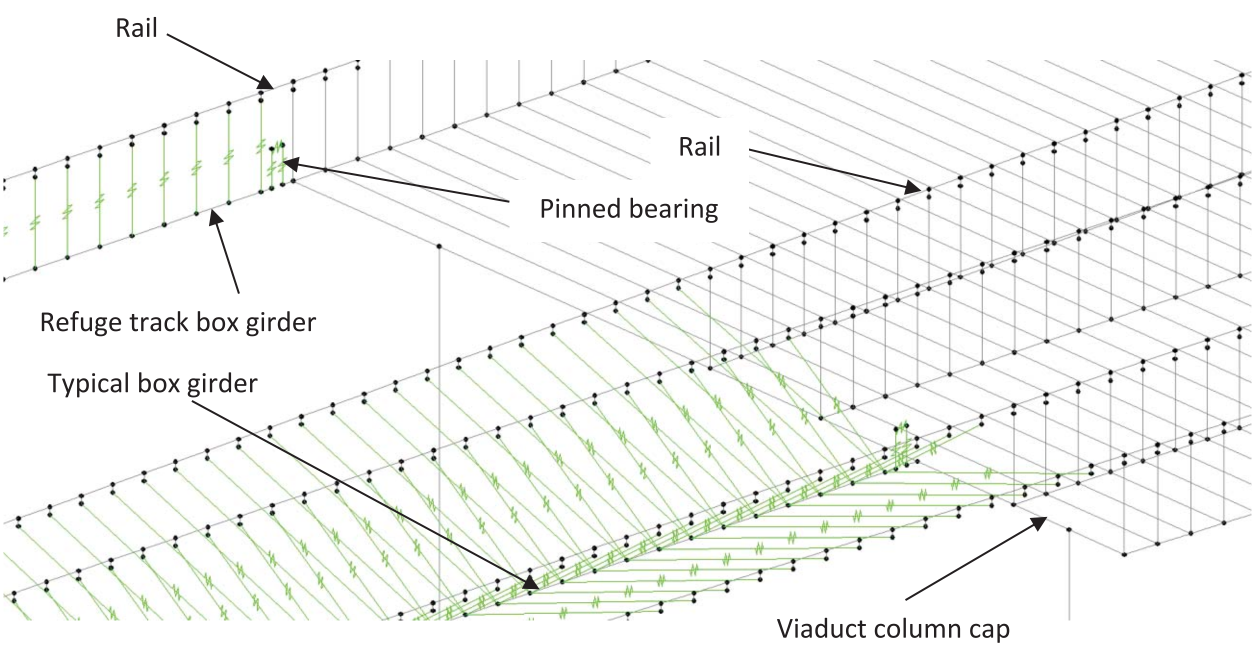
SAP Model: 3D View Hanford Station



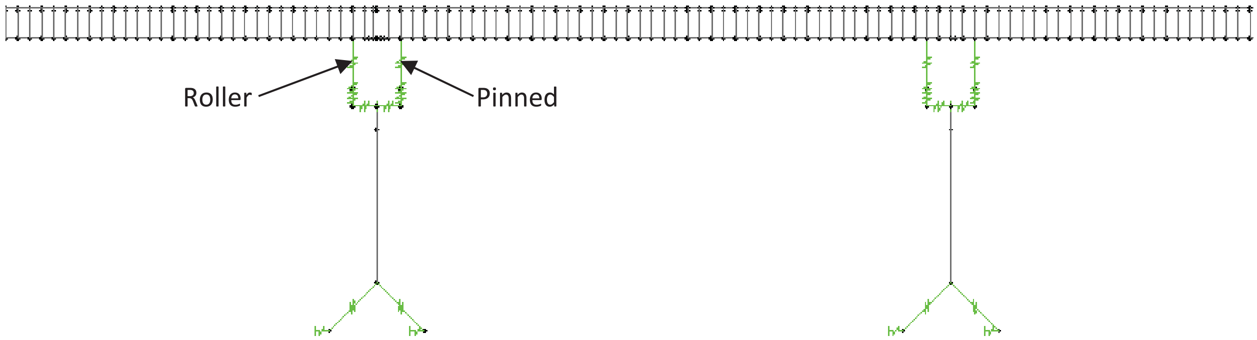
SAP Model: 3D View Expansion Joint



SAP Model: Joint Detail

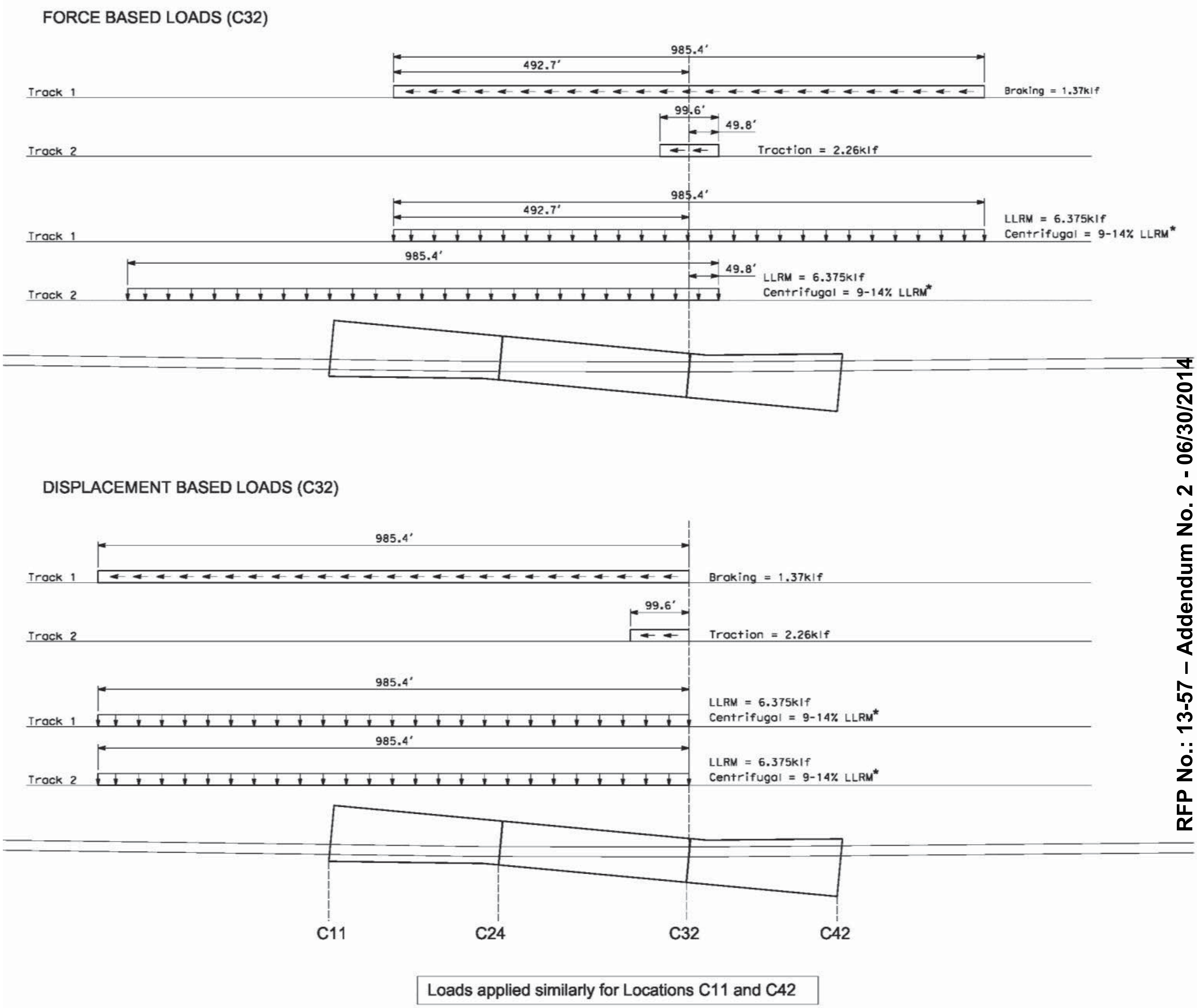


SAP Model: Transition Joint Detail



SAP Model: Viaduct Articulation

- Joint load application example (from Conejo Crossover): Conejo Crossover, Corcoran Crossover, Poso Creek Crossover



*Centrifugal forces are applied at 8ft above rail level and act perpendicular to the rails.

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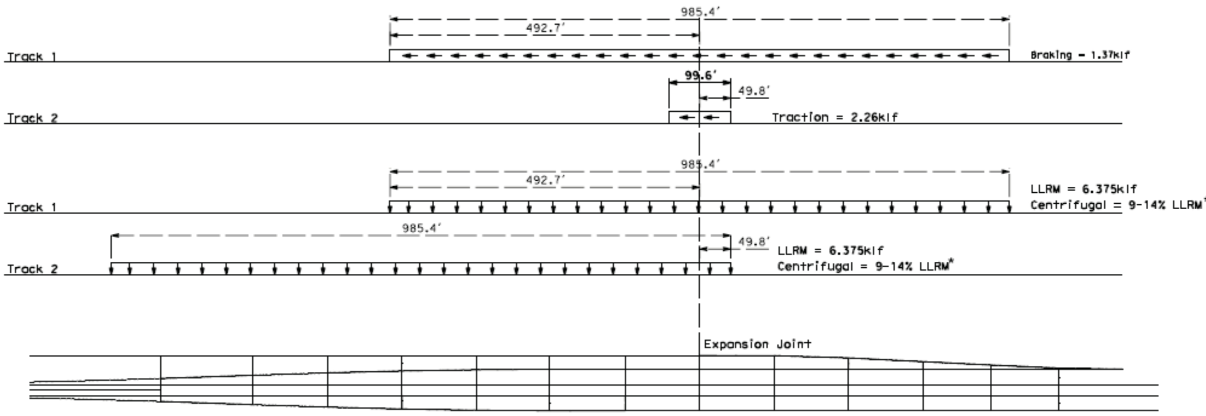
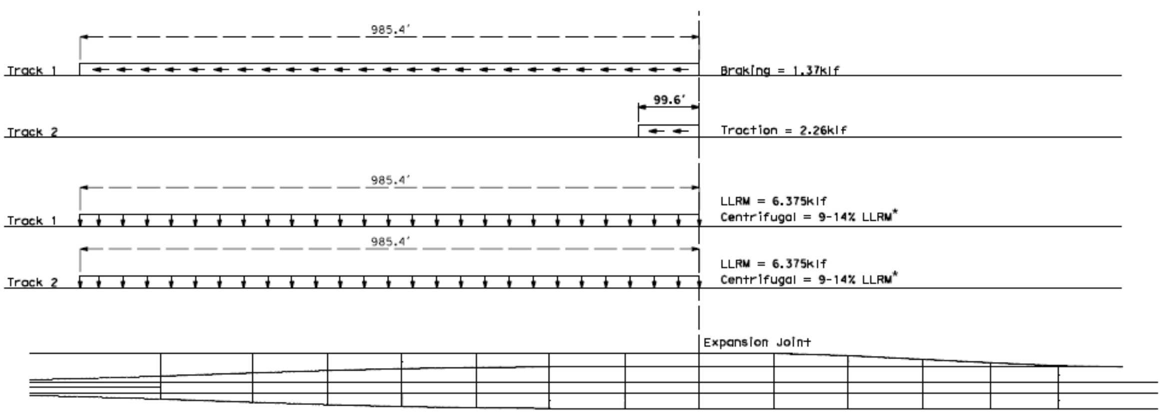
- Joint load application example: Hanford Station

FORCE BASED LOADS (C32)



DISPLACEMENT BASED LOADS (C32)



Loads applied similarly for other expansion joint locations

*Centrifugal loads are applied horizontally and perpendicular to the rails

- Train mass has been represented, if required by analysis. This is applied to joints 8' above TOR, connected to superstructure element with rigid links
- Stiffness and mass modeling:

Analysis	Stiffness	Mass
Frequency Analysis (TM 2.10.10 Section 6.4)	Upper Bound: I_g with 1.3x nominal f'_c ($1.14E_{nominal}I_g$) Lower Bound: I_{cr} with nominal strength ($0.3E_{nominal}I_g$)	Lower Bound: 0.95 Upper Bound: 1.05
Component Force Design under OBE loads	Upper Bound: I_g with 1.3x nominal f'_c ($1.14E_{nominal}I_g$) [for maximum force distributed to columns and piles]	Nominal: 1.0
Train-Structure Interaction Groups 4 and 5	Lower Bound: I_{cr} with nominal strength ($0.3E_{nominal}I_g$) [for maximum displacement and rail stress]	Upper Bound: 1.05